FACTORS INFLUENCING LABORATORY VIBRATORY COMPACTION

by

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To my Father and Mother

DECLARATION OF CANDIDATE

I , Jan J Troost, hereby declare that this thesis is my own work and that it has not been submitted for a degree at another university.

Signed by candidate

Signature Removed

J J Troost
September 1987

SYNOPSIS

The thesis consists of a literature review and a limited experimental investigation in a soils laboratory.

The objective of the literature review is to determine what standard laboratory test methods based on vibration exist for the control of compaction, to what soil types these tests are applicable and what the factors are which affect laboratory vibratory compaction.

The study revealed that extensive research has been carried out in the USA and Europe, where standard laboratory compaction tests exist for the determination of the maximum dry density of cohesionless, free-draining soil. The US methods are based on the use of a vibratory table, while the European practice is based on the use of a vibratory tamper. No standard tests appear to exist for soil exhibiting cohesion, though limited research has been carried out in the USA into the behaviour of such soils under laboratory vibratory compaction.

The factors; frequency, amplitude, mould size and shape surcharge intensity and manner of application, soil type, time of vibration, number of layers and moisture content are all reported to have an effect on the maximum dry density achievable.

It has been recognised that significant interaction occurs between the factors affecting vibratory compaction, but the extent of the interaction appears to be only partly understood.

The objective of the limited experimental program was to determine whether a specific graded crushed stone could be compacted to Modified AASHTO maximum dry density with a laboratory vibratory compaction technique using a vibratory table, and how this could best be achieved.

The effects on dry density of changing the frequency, the time of vibration, mould size, surcharge pressure, grading and moisture content were investigated.

It is concluded that the graded crushed stone in question can be compacted to Mod. AASHTO maximum dry density but that before reliable reproducible results can be achieved with this type of test further work is necessary. Such research should be aimed at investigating the interaction effect between the amplitude of vibration, the soil type and the type and intensity of the applied surcharge pressure.

ACKNOWLEDGEMENTS

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The assistance of Mr D Smith and his staff at the Ninham Shand laboratory is greatly appreciated. In particular I would mention Mr R Smith and Mr K Zaal who carried out the soil classification tests; Mr V Masias and Mr G Williams who performed the Modified AASHTO density determinations and Messrs H Makina, P Manyela, A Petrus and E Valentine who assisted with the hard work of sieving and drying the soil samples.

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The soil samples for the experimental work were kindly supplied by Peak Quarry (Cape) (Pty) Ltd and from the Eersterivier Plant of Much Asphalt (Pty) Ltd.

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ASTM* Designation:D 653-86. Standard terms and symbols relating to soil and rock

ASTM* Designation:D 698-78 (-42T, -70). Standard test methods for moisture-density relations of soils and soil-aggregate mixtures using 5.5-lb (2.49-kg) rammer and 12-in. (305-mm) drop.

ASTM* Designation:D 1557-78 (-58T, -70). Standard test methods for moisture-density relations of soils and soil-aggregate mixtures using 10-lb (4.54-kg) rammer and 18-in. (457-mm) drop.

ASTM* Designation:D 2049-69. Standard test method for relative density of cohesionless soils.

ASTM* Designation:D 4253-83. Standard test methods for maximum index density of soils using a vibratory table.

ASTM* Designation:D 4254-83. Standard test methods for minimum index density for soils and calculations of relative density.

* An ASTM designation consists of a series of numbers e.g. D 698-78 (-42T, -70). Of these the first i.e. D 698 is the number of the test and the second i.e. 78 indicates that the latest revision was in 1978. The numbers in brackets indicate that the standard was first published in 1942 and later updated in 1970.

AASHTO** Designation:T99-81. Standard method of test for moisture-density relations of soils and soil aggregate mixtures using 5.5lb (2.49kg) rammer and 12in (305mm) drop.

AASHTO** Designation:T180-74 (1982). Standard method of test for moisture density relations of soils and soil-aggregate mixtures using 10lb (4.54kg) rammer and 18in (457mm) drop.

** The two abbreviations stand for the following:

AASHO - American Association of State Highway Officials
AASHTO - American Association of State Highway and Transportion Officials

These are two names for the same body, the name change from AASHO to AASHTO having come into effect in 1971. The change was in name only, and the terms are used interchangeably in the text. i.e. Modified AASHO density is the same as Modified AASHTO density.

BS 1377 : 1975. Methods of test for soils for Civil Engineering purposes.
Test 12. Determination of the dry density/moisture content relationship (2.5kg rammer method).
Test 13. Determination of the dry density/moisture content relationship (4.5kg rammer method).
Test 14. Determination of the dry density/moisture content relationship of granular soil (vibratory hammer method).

TMH1-1979*** Standard methods for testing road construction materials. Method A7. The determination of the maximum dry density and optimum moisture content of gravel, soil and sand.

TMH1-1982*** (Supplement). Standard methods for testing road construction materials.
Method A11T. Tentative method for the determination of the maximum density and optimum moisture content of graded crushed stone and cohesionless sand by means of vibration compaction.

TMH1-1986*** (Second edition). Standard methods of testing road construction materials.
Method A11T. Tentative method for the determination of the maximum dry density and optimum moisture content of graded crushed stone and cohesionless sand by means of vibration compaction.

*** The Technical Methods for Highways (TMH) series is published by the South African National Institute for Transport and Road Research on behalf of the Committee for State Road Authorities and serve as manuals for engineers, prescribing standard methods to be used in road construction.

LIST OF PRINCIPAL SYMBOLS AND ABBREVIATIONS USED

a_{max}	peak acceleration (m/s^2) (cf Appendix A.4)
d	particle size or sieve opening size (mm)
d_{max}	maximum particle size in a sample (mm)
d_{60}	particle diameter at 60% of cumulative particle size distribution (mm)
d_{10}	particle diameter at 10% of cumulative particle size distribution (mm)
e_{max}	maximum void ratio determined in the laboratory
e_{min}	minimum void ratio determined in the laboratory
f	frequency (Hz)
g	acceleration due to gravity (m/s^2)
k	coefficient of permeability (cm/s)
n	index $0.5 < n < 0.3$ used in the Talbot equation (cf Appendix A.1)
w	moisture content
A	amplitude = ½ double amplitude (cf Appendix A.4)
AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
AD	apparent density (kg/m^3) (cf Appendix A.8)
ASTM	American Society for Testing and Materials
BS	British Standard
CBR	California Bearing Ratio

C_u coefficient of uniformity d_{60}/d_{10} (cf Appendix A.2)

D_r relative density (cf Appendix A)

D_{max} maximum index density determined in the laboratory (kg/m³)

D_{min} minimum index density determined in the laboratory (kg/m³)

Gs apparent relative density (cf Appendix A.B)

LL liquid limit

MADD maximum dry density achieved with modified AASHTO compactive effort (kg/m³)

MVDD maximum dry density achieved with vibratory method (kg/m³)

OMC optimum moisture content (%)

OMCA optimum moisture content under modified AASHTO compactive effort (%)

OMCV optimum moisture content under laboratory vibratory compaction (%)

P percentage passing a given sieve size. (cf Talbot equation, Appendix A.1)

PI plasticity index (i.e LL - PL)

PL plastic limit

S_{rv} 'specific rugosity' (a measure of the surface roughness of an individual stone or soil particle (ref v.d. Merwe, 1984))

SVF specimen volume factor (cf Section 3.4.6.1)

USBR United States Bureau of Reclamation

γ_d dry density or field dry density (kg/m³)

1. INTRODUCTION.

This thesis is concerned with determining, through literature survey and experiments, the significant factors affecting the compaction of soil by vibration in the laboratory.

Vibration as a means of compacting soil in the field has been in use since the early 1930's. Prior to 1960, such vibration was only applied to cohesionless soils, but with improvements in compaction plant vibration is used today to compact not only cohesionless soils such as sand and gravel but also rock fill, soil cement, silt and clay. (Forssblad, 1980). Laboratory control of compaction is, however, still largely based on non-vibratory methods.

In the USA and Europe where the relative density formula is used for the control of compaction of cohesionless, free-draining soil a series of standard laboratory compaction tests based on vibration exist. The development of these tests and the research into the factors affecting vibratory compaction in the laboratory are discussed in Sections 2.4 and 2.5.

No standard test methods based on vibration appear to exist for soils exhibiting cohesive and non-free draining properties. Some research has however been carried out in the USA into laboratory vibratory compaction of soil with some cohesion. This research is reviewed in Section 2.6.

A limited number of experiments to determine the behaviour under vibratory compaction of a graded crushed stone were carried out in a soils laboratory. These were undertaken as part of the research for this dissertation and are presented in Section 3.

2. LITERATURE REVIEW.

2.1 Historical Development of Vibratory Compaction in the Field.

2.1.1 General.

The development of compaction in general, but especially vibratory compaction, is inextricably linked to the history of road construction. Compaction has also found important applications throughout history in the construction of earthfill dams and other water retaining structures, as well as structural foundations. But it is in the field of road construction where vibratory compaction found its first application in the field in pre-war Germany. It is appropriate therefore to trace the history of compaction as it developed in road construction since Roman times, to gain a perspective as to the origin and need for vibratory compactive techniques. Figure 2.1 shows some of the earlier methods of road construction which are referred to in the text.

2.1.2 The roads of Ancient Rome.

In the first century BC the Roman, Vitruvius, made one of the first references to compaction of earthworks in road construction, when he wrote in a discussion of soils in Book II of his ten books on Architecture:

> "When the mass has been spread, ten men should ram it with rammers. This ramming should continue until the mass is solid .... and compressed to three quarters of its initial height".

This method of ramming for layerwork appears to have been the most common method of compaction for the mechanical stabilization of the vast road networks built by the Romans. These layerworks were placed on a hand-placed layer of larger stones.

2.1.3 The Middle Ages.

After the collapse of the Roman Empire, engineering skills declined, communications and commerce reduced to a low level and most travel was on foot or horseback. The existing roads deteriorated and few new roads were built until the early 18th Century.

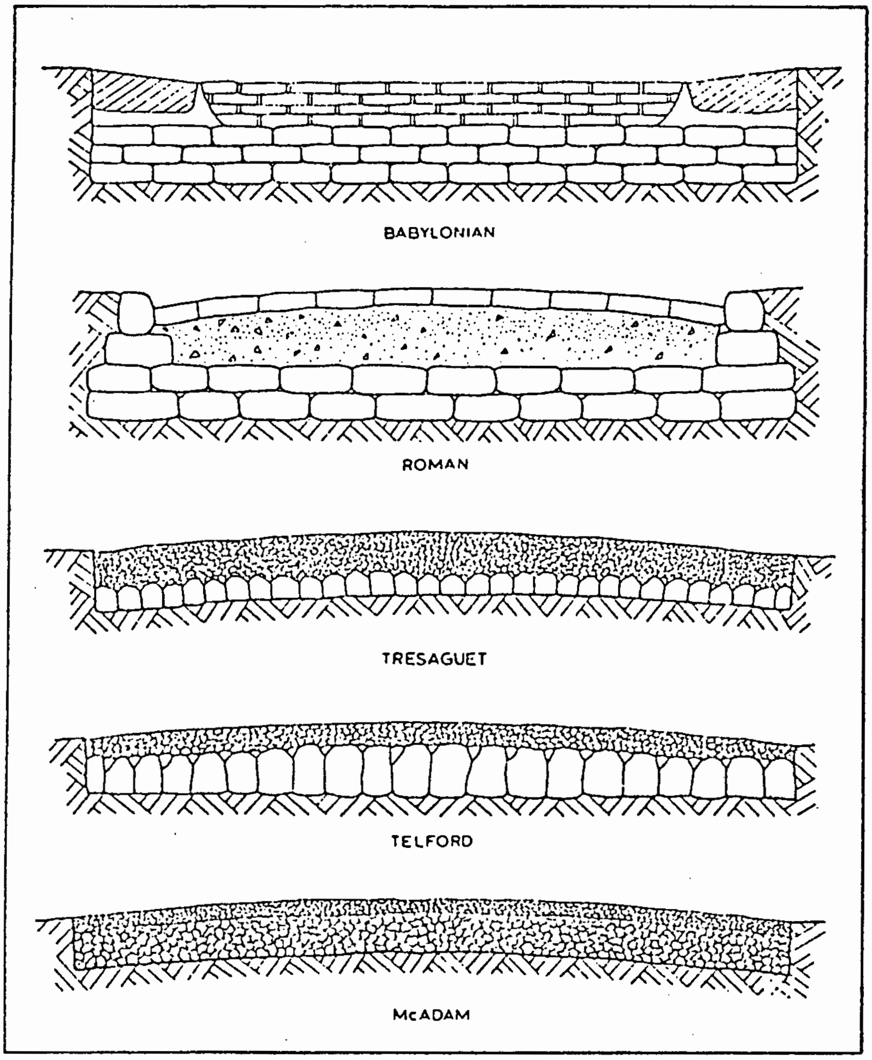


FIGURE 2.1 :- ROAD CROSS—SECTIONS SHOWING DEVELOPEMENT OF CONSTRUCTION METHODS (AFTER SCHWARTZ, 1978)

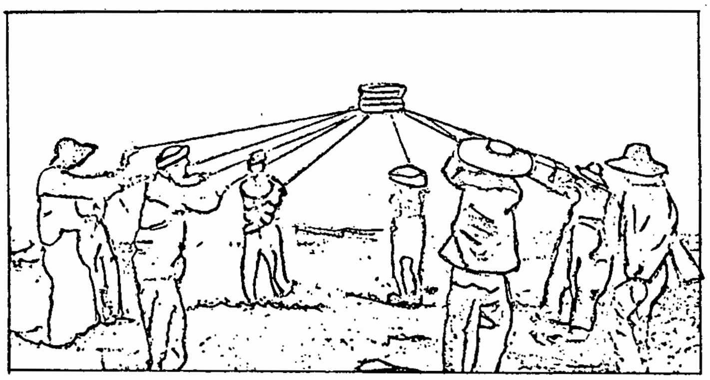


FIGURE 2.2 :- METHOD OF DYNAMIC SOIL COMPACTION BY TAMPING USED IN CHINA. (ENGINEERING NEWS RECORD, OCT. 1949)

2.1.4 The start of modern compaction principles.

The use of broken stone to provide a stable road base was pioneered in France in the late 18th Century by Tresaguet. In the early 1800's similar principles were applied by British engineers such as, Telford and McAdam, in England. These roads depended, for their stability on aggregate interlock and the principal design task was to provide adequate drainage. Since the roadstones were large and were carefully packed by hand on adequate subgrade there was very little need for mechanical compaction.

With the development of the roads and increased traffic density (still mostly horsedrawn carriages and wagons) mechanical compaction became a necessity. This resulted in the development of the steamroller in the early half of the 19th century, and with it, there was a growing realization that moisture in the soil played an important role in compaction. At this time rolling was however limited to the existing ground surface and no special effort was made to compact highway embankments. The road surfaces were flexible enough to remain trafficable despite limited settlement of the fill.

The first "sheepsfoot" roller, consisting of a drum with hundreds of short stubs protruding, was invented in the early 1900's. This device was horsedrawn and weighed approximately 2.5 tonnes. (cf A typical 1987 Dynapac vibratory tandem roller can weigh up to 15 tonnes).

2.1.5 The developments from 1918 to 1945.

With the advent of motorized transport after World War I, there were rapid developments in compaction techniques. A new higher standard of road was required, capable of supporting freight traffic of up to 6 tonne motor vehicles travelling at some 30 km/h. Until 1914 most freight traffic had been limited to 1.5 tonne horsedrawn wagons.

By the 1920's many smooth wheel steam rollers had been fitted with petrol engines. Some of these rollers had masses of up to 15 tonnes.

The first compaction machinery utilizing vibration was developed in Germany during the 1930's. A self-propelled vibratory plate compactor and a 25 ton crawler mounted vibratory compactor were built.

During World War II, the US Army Corps of Engineers used possibly the first self-propelled and tractor-towed vibratory rollers.

2.1.6 The advent of vibratory compaction machinery.

In the 1950's there were significant developments in the use of vibration for compaction of cohesionless and free-draining materials and vibratory compaction machinery evolved rapidly. The initial developments were in towed smooth-drum vibratory rollers of 4 to 6 tonnes. In the early 1960's towed vibratory sheepsfoot rollers for clay and other cohesive soils were introduced, and since about 1970 self-propelled vibrating rollers have been in use.

Self-propelled vibratory rollers have become increasingly more versatile with modern tandem rollers having vibration on both drums. In addition an extensive range of walk-behind single and double drum rollers, plate compactors and tampers with vibratory characteristics have found widespread use. According to a manufacturer some 70% of compaction equipment sold today has vibratory characteristics. (Forssblad, 1980)

2.2 Historical Development of Vibratory Laboratory Compaction Tests.

2.2.1 Before the Proctor test.

The use of vibration in laboratory compaction tests started in the early 1950's, at the time of the advent of vibratory compaction in the field. In order to illustrate the significance of the use of vibration in laboratory compaction, it is necessary to consider the developments in laboratory compaction testing before 1950.

For the compaction of road material in Ancient Rome, Vitruvius' adage, ".... ramming should continue until the mass is solid ..... and compressed to three quarters of its initial height", was in effect a compaction specification. This rule of thumb may well have been the most scientific method of controlling compaction in use, up until the twentieth century.

The development of powered excavating and hauling equipment that followed the invention of the internal combustion engine at the end of the 19th Century resulted in marked increases in the heights of fills. These fills were constructed by means of end-tipping and the loose soil was not mechanically compacted but allowed to "settle". Paving was delayed until the fill had had sufficient time to consolidate, often a matter of years.

From the early 1920's onwards, traffic volumes increased dramatically and demand increased for a shortened time interval between the conclusion of earthwork construction and the commencement of paving. Consequently soil was placed in layers and compacted systematically. The compaction requirements, which for obvious reasons often led to controversy included such terms as "thoroughly compacted" or "compacted to the satisfaction of the engineer". (Schwartz, 1978).

The first attempts at investigating methods of compacting soil samples at varying moisture contents in order to control compaction in the field appear to have been made in 1929 by the California Division of Highways. This work developed into what became known as the Proctor Test. Proctor, who was employed by the Bureau of Waterworks and Supply of the City of Los Angeles, proposed a specific compaction test method which was developed for use in earth dam construction.

The apparatus developed by Proctor consisted of a cylindrical container 102mm (4-in) diameter and 127mm (5-in) deep. Soil was compacted in three layers by a 2.5 kg (5.5-lb) rammer with a diameter of 51mm (2-in) by subjecting each layer to 25 strokes of the rammer.

Following the publication of Proctor's test method in 1933 (Proctor, 1933), US highway authorities developed their own version of the test. Some agencies made changes in the number of layers, the size of the container and the compactive effort.

A "standardized" version of the Proctor test was introduced by the American Association of State Highway Officials (AASHO, later AASHTO cf list of standard test methods) in 1938 and was published as AASHO Designation T99-38, and by the American Society for Testing Materials in 1942 as ASTM Designation: D698-42T. In the standardized test the original 25 firm 305mm "strokes" per layer became 25 blows from the rammer dropping freely from 305mm (12-in). This standard compactive effort applied to three layers is still referred to as "Standard AASHTO" compactive effort.

In the early 1940's the US Army Corps of Engineers developed what became known as the Modified AASHTO method. This test employed a 152mm (6-in) diameter by 127mm (5-in) high compacted specimen built up in 5 layers, each subjected to 55 blows of a 4.5kg (10-lb) rammer with a free drop of 457mm (18-in). The test was standardized as AASHO Designation T180 in 1957 and ASTM Designation: D1557-58 in 1958.

Although the AASHO and ASTM standards had been introduced, there were many variations used by other authorities. Some of these are presented in Table 2.1, where it may be seen that the compactive energy per unit volume more or less corresponds to the Standard and Modified AASHO methods. Parameters such as mould size and maximum particle size however, varied from one test to another.

These differences were probably introduced only for convenience (e.g use of available equipment). Later research showed that even small changes in mould dimensions and maximum particle size as well as the type, magnitude and distribution of the compactive effort applied, had significant effect on the density achieved. These factors are discussed in more detail in 2.4.

The South African standard test method for determining maximum dry density and OMC under Modified AASHTO compactive effort, TMH1-A7 (cf Table 2.1), uses a mould diameter and specimen height which differ from those specified in Modified AASHTO Designation T180-74. Because confinement differs, the two test methods are likely to yield different "maximum" dry densities and OMC's for the same soil.

	Standard AASHO (T99) ASTM D 698 Method A and C	Modified AASHO (T180) ASTM D 1557 Method A and C	Bureau of Reclamation, USA	Army Corps of Engineers, USA	British Standard BS 1377		German Standard Din 18127		South African Standard TMH 1
Mould									
Diameter mm	102	102	108	152	105	105	100	100	152
Height mm	116	116	152	114	115.5	115.5	120	120	127
Volume cm^3	944	944	1416	2082	1000	1000	942	942	2304
Rammer									
Weight kg	2.49	4.54	2.49	4.54	2.50	4.50	2.50	4.50	4.54
Drop height mm	305	457	457	457	300	450	300	450	457
Diameter mm	51	51	51	51	50	50	50	50	51
Layer									
Number	3	5	3	5	3	5	3	5	5
Material									
Maximum particle size mm	A:4.75 C:19.1	A:4.75 C:19.1	4.75	19.1	20	20	20	20	19
Compaction effort									
Blows per layer	25	25	25	55	25	25	25	25	55
Energy Nm/m^3	$5.9 \cdot 10^5$	$2.7 \cdot 10^6$	$5.9 \cdot 10^5$	$2.7 \cdot 10^6$	$5.5 \cdot 10^5$	$2.5 \cdot 10^6$	$5.9 \cdot 10^5$	$2.6 \cdot 10^6$	$2.4 \cdot 10^6$

TABLE 2.1 :- STANDARDIZED PROCTOR-TYPE TEST METHODS (AFTER FORSSBLAD, 1971)

As a corollary to this, the OMC determined by a specific laboratory test may be significantly different from that in the field, since the confinement condition and mode of compaction may differ substantially in the two cases.

Due to the larger compactive effort used in the Modified AASHTO test, the maximum dry density that was achieved was some 5% to 10% higher for granular soils than with the Standard AASHTO method. The optimum moisture content (OMC) was found to be typically 3 to 8% lower with Modified AASHTO than with Standard AASHTO compactive effort.

Despite the apparent difficulty of providing a laboratory "Proctor type" test standard, the procedures implied in the method have been shown to have value in standardising compaction for a wide range of materials used in construction. A notable exception to this is the compaction of coarse granular materials such as gravels, sands and graded crushed stone such as that used in the construction of base course, for which the Proctor type tests have been shown to be unsuitable. These materials appear to additionally require vibration to provide the particle rearrangement required for high densities.

2.2.2 Laboratory vibratory testing in the USA.

In 1954 a subcommittee was appointed by the ASTM to determine methods in the laboratory of achieving standardized maximum and minimum densities for cohesionless materials. These methods were to provide a means for controlling compaction based upon either "percent compaction" or "relative density" specifications (cf Appendix A.9). The chairman of this subcommittee, Earl J Felt (1958), gave the following reasons as to why the then current Standard AASHTO moisture density test (ASTM Designation: D698-57) was unsuitable for cohesionless soils:

* The dynamic compaction as used in the Proctor test methods, employing a rammer smaller in size than the confining mould, resulted in cohesionless sand displacing when struck by the rammer.

* With coarse-graded crushed stone the method was ineffective because the inherent angular stability of the particles prevented proper densification. For these materials there was insufficient opportunity for the particles to move horizontally into closer orientation.

* Repeated ramming resulted in degradation of the material.

* Some nearly cohesionless soils compacted satisfactorily in the Proctor tests but the moisture-density curve was not well defined and the indicated maximum density (Standard AASHO, pre-1958) was not as great as could be achieved readily in the field.

In addition Felt quoted Burmeister (1948) who had advocated that "relative density" was a more satisfactory index of soil shear strength than simply "density". Relative density had also been correlated with other physical properties of soils. These included angle of internal friction (Wu, 1957), bearing pressure (D'Appolonia, 1953), permeability (Jones, 1954) and triaxial shear strength (Holtz and Gibbs, 1956). The history of vibratory laboratory compaction therefore became closely linked to the concept of relative density.

The "maximum density" which the Sub-committee sought to achieve in the laboratory was the "absolute maximum" for any particular soil. This was not to avoid the seemingly ironical situation of specifications calling for greater than 100% AASHO maximum dry densities on a percentage basis, but rather to accommodate the use of control based on the Relative Density formula.

Research showed, without doubt, that it was necessary to provide vibration in the compaction of coarse granular materials in order to achieve high densities.

The work of the Sub-committee, initiated in 1954 on the compaction by vibration in the laboratory, led to the publication in 1969 of ASTM Designation: D2049-69 entitled "Standard test methods for relative density of cohesionless soils".

The vibratory method in ASTM 2049-69 for determining the "maximum" density, applied to "cohesionless, free-draining soils for which impact compaction will not produce a well-defined moisture-density relationship curve and the maximum density by impact methods will generally be less than by vibratory methods". These soils could contain up to 12% by weight of soil particles passing a 0.075mm sieve, provided they are still free-draining.

The components of the apparatus for ASTM 2049-69 were specified. Vibration was to be imparted by a vibratory table with a cushioned steel vibratory deck about 30 in. by 30 in. (762mm x 762mm) actuated by an electro-magnetic vibrator. The net weight of the vibrator was to be over 100 lb (45.5 kg). A frequency fixed at 60 Hz was specified and the amplitude was to be variable over the range 0.002 to 0.025 in (0.05mm to 0.64mm) under a 250 lb (113 kg) load.

A 0.5 ft^3 (14 160 cm^3) mould was specified for soil having a maximum particle size of 3 in (76mm) and a 0.1 ft^3 (2 830 cm^3) mould was to be used for soils with maximum particle sizes of 1½ in (38mm) and less.

Samples were to be vibrated at maximum amplitude for 8 minutes under a solid mass surcharge of 2 psi (14 kPa).

Soil had to be tested in the oven-dry or wet condition and the highest result taken as the "maximum" density. For the "wet" test sufficient water was required to allow a small amount of free water to accumulate on the surface during filling of the mould. (cf Appendix A.7).

Problems experienced with the use of the "relative density" concept were discussed at a Symposium at the 75th Annual Meeting of the ASTM in 1972. The symposium was entitled:

> "Evaluation of relative density and its role in geotechnical projects involving cohesionless soils."

At the Symposium, Tavenas (1972), contended, on the basis of results from comparative tests, that the maximum and minimum densities of cohesionless soils could not be accurately measured by ASTM D 2049-69. He also held that as a result of this, published correlations of relative density with the mechanical properties of the soil are a function of the laboratory performing the control tests.

The symposium as a whole felt however, that the relative density concept had merit in expressing general trends of performance, but that it could not be regarded as superior to other methods for compaction control. They concluded that the physical properties of cohesionless materials were not only a function of density but also of size, grading, shape and angularity of particles. (Selig and Ladd, 1972).

It was recognized that for the vibratory test the optimum combination of frequency and amplitude depended on the soil and that such factors as mould size, surcharge and mode of vibration had a significant effect on the density achieved.

Although the concept of relative density had become a controversial one, vibratory compaction was still regarded as the most suitable way of compacting cohesionless, free-draining material in the laboratory. Consequently a new test entitled "Standard test methods for the maximum index density of soils using a vibratory table" was published in 1983 with ASTM Designation: D4253-83.

The test aims at providing <u>a</u> maximum density and not <u>the</u> maximum density for a soil and allows for different methods of test. In the preamble to the test it is stated that the individual assigning the test should specify the test method. The influence of a test method is thus recognized, and due cognisance of such method should be taken when interpreting results.

The test methods are applicable to soils in which 100%, by dry weight, of soil particles pass a 3-in (75mm) sieve and which may contain up to 15% of soil particles passing the 0.075mm sieve, provided they still have cohesionless, free-draining characteristics, and 30% of soil particles are retained on a 1.5-in (37.5mm) sieve.

The four alternative procedures involve testing either oven-dried or wet soil on either an electromagnetic or cam-driven vertically vibrating table. Samples are vibrated under 2 psi (14 kPa) dead mass surcharge.

Testing may be carried out at a double amplitude (cf Appendix A.3) of 0.013 in (0.33mm) for 8 min at 60 Hz or at 0.019 in (0.48mm) for 10 min at 50 Hz. Furthermore it is recognized that for a given frequency of vibration a soil may reach a peak density at an optimum double amplitude. For this reason the double amplitude may be varied.

Standard moulds of a 0.1 ft^3 (2 830 cm^3) and 0.5 ft^3 (14 160 cm^3) are specified as in ASTM 2049-69. In addition special moulds with diameters between 70mm and 100mm may be used for special studies (e.g. triaxial testing).

The test procedures allow the variation of a number of parameters in an endeavour to achieve the highest possible density with a soil. The unique characteristics of a soil are thereby recognized and the need to quote the exact testing procedure with the results is underscored.

This test method reflects the current understanding of laboratory vibratory compaction.

2.2.3 Laboratory vibratory testing in Europe

In Europe independent research into laboratory vibratory compaction has been carried out. The aim of the tests has been, as in the USA, to achieve a maximum density for cohesionless soil, for specifications using the relative density formula.

Rather than use a vibrating table, the Europeans have favoured a vibrating hammer or tamper which introduces vibration from the top while the mould is secured to a fixed base. The two types of methods are illustrated in Figure 2.3.

One method using a vibratory tamper has been developed in Sweden. (Forssblad, 1967). A similar method, using a vibratory hammer was developed in England, and has been adopted as BS 1377, Test 14.

The British test, BS1377, Test 14 (1975) is suitable for fine-grained granular soils and for the fraction of medium- and coarse-grained granular soils passing the 37.5 mm sieve. The soil is tested over a range of moisture contents and not only "wet" or "dry" as in the ASTM procedures.

Compaction is in three layers in a California Bearing Ratio mould of 152mm diameter and 127mm depth. Vibration is imparted by an electrically operated vibrating hammer at a frequency between 25 and 45 Hz. The steel tamper 145mm in diameter is limited to a mass of 3kg. Each layer is compacted for 60 sec under "firm downward pressure". The stroke of the tamper is not specified.

In Sweden, ASTM Designation: D4253-83 has been introduced in the specification of crushed rock road base course materials. A 10-in (254mm) diameter mould is used. (Forssblad-personal communication 1987).

2.2.4 Laboratory vibratory testing in South Africa.

In 1982 the National Institute for Transport and Road Research of the CSIR published the test method TMH1-A11T as a supplement to TMH1 of 1979. The test was entitled "Tentative method for the determination of the maximum dry density and optimum moisture content of graded crushed stone and cohesionless sand by means of vibration compaction".

The test is for determining maximum dry density for the purposes of specifying compaction on a percentage basis only. In contrast to overseas practice the use of the relative density method of compaction control is seldom used in South Africa.

The method is for "graded crushed stone" and "cohesionless" sand, but no reference is specifically made to free-draining characteristics.

The maximum particle size is 37.5mm and material is compacted in a single size mould 152mm in diameter to a total depth of 127mm. Compaction takes place on a vibratory table (size and type not specified) with a fixed amplitude of 1 $\pm$ 0.5mm at a frequency of 47 $\pm$ 3 Hz. (cf Appendix A.3 - amplitude).

The sample is compacted in 2 layers, for 2 minutes per layer, under a solid mass surcharge of 50 kg (27 kPa). Soil samples are compacted over a range of moisture contents to determine the OMC.

The test method has been republished in an edited form, but still under the same title, in the second edition of TMH1 (1986). In this revised version the sample is compacted in 3 layers for 2 minutes per layer.

This South African method TMH1-A11T, (1986) was used as the starting point for the experimental work described in Section 3.

2.3 Types of soil to which standard laboratory vibratory methods apply.

The properties, "free draining" and "cohesionless", have been mentioned in Section 2.2.2 as pre-requisites for a material to be suitable for standard laboratory vibratory compaction. This appears to be largely due to the fact that the standard vibratory test methods have invariably been linked to the relative density approach for controlling compaction. A soil has been classified as suitable either for compaction control by an impact method or the relative density method. For borderline cases both impact and relative density approaches has been used and the more suitable method selected on the basis of which density was higher, 95% standard proctor or 70% relative density. The ASTM Test for Relative Density of Cohesionless Soils (ASTM Designation: D2049-69) suggests that a maximum of 12% fines be used as a rough guide to judge whether or not a soil is free-draining.

It is important to note that at the time of publication in 1969 of ASTM D:2049-69 the mechanism of laboratory vibratory compaction was not well understood. Moreover comparisons were made with Standard AASHO maximum dry density rather than Modified AASHO maximum dry density, which is from 5 to 10% higher.

Specification on the applicability of laboratory vibratory compactions was also given by Holtz, (1957 and 1972). Holtz states, that the soils to which laboratory vibratory compaction are applicable can be grouped according to the Unified Classification System (see Figure 2.13) as follows:

1. GW, GP, SW, SP soils on suitable. (The fines are limited to 5% by definition). (cf Appendix A.5 - fines).

2. Borderline GW-GC, GW-GC, GP-GM and GP-GC soils containing less than 8% fines are usually suitable.

3. Borderline SW-SM, SP-SM and SP-SC soils are suitable (fines are limited to 12% by definition).

4. SM and SC soils require special consideration and suitability depends upon gradation of the sand and the plasticity of the fines. Some SM soils with up to 16% fines have proved suitable.

All the above soils are classified as "coarse-grained" for which more than 50% of the material is larger than 0.075mm. SM contains non-plastic fines only but SC contains plastic fines.

The standard maximum test for soils using a vibratory table, ASTM D4253-83 of 1983, allows up to 15% fines but stresses that material must be free-draining and cohesionless.

There exists however a discrepancy between the soils to which the vibratory test is said to be applicable and the concepts "free-draining" and "cohesionless". SC and SM sands for example are considered impervious and contain in excess of 12% fines.

The USBR (Earth Manual, 1955) however showed that there is a poor correlation between permeability and the effectiveness of vibratory compaction. Also Nettles and Calhoun, (1967) showed that at Modified AASHTO density, materials with more than 5% fines are only marginally permeable.

Townsend (1972) carried out tests on SP and SP-SW sands adding up to 23% of plastic fines with a PI = 12. He found that these soils compact satisfactorily under vibration (compared with Standard AASHTO maximum dry density) although the soils are by no means either cohesionless or free-draining. Moisture and plasticity were however found to be interrelated factors which greatly affect compaction. The experiments carried out by Townsend and others on materials with some cohesion are discussed in more detail in Section 2.6.

2.4 Factors influencing laboratory compaction.

2.4.1 General.

Since 1933, when the first Proctor-type tests were devised for laboratory compaction, studies have been undertaken to assess the factors which influence compaction of soil in the laboratory. An examination of the available literature reveals that it has not always been recognized that those principles affecting compaction in the laboratory are not necessarily the same as those which apply in the field. An example of this is the optimum moisture content (OMC). The OMC is related to a specific procedure in the laboratory in which compactive energy is applied by a given technique. In the laboratory it is recognized that if the compactive effort is changed the OMC also changes. This principle is graphically illustrated in Figure 2.4. The relationships between dry density and moisture content are for the same soil, a sandy clay (LL = 29 and PI = 7). The lower curve shows the dry density/moisture content relation for the soil compacted by applying 25 blows per layer on each of three layers according to BS 1377 Test 12, and the upper curve the corresponding relationship when 100 blows per layer were applied. The OMC for the lower compactive effort is approximately 15% while for the higher effort it is 12%, some 3% lower for this particular soil. Despite this, it is often assumed that in the field, regardless of the specific characteristics of the compaction plant (i.e. the medium chosen to apply compactive effort), the OMC is that indicated by the laboratory test selected to control the compaction.

While assessing the influence of any one factor on compaction in the laboratory the above principle applies. If, for example the size of the mould is found to affect the density achieved with a given impact test, the effect need not necessarily be the same if another mode of compaction (e.g. vibratory compaction), is used.

The factors which affect laboratory compaction are divided into two groups for the purposes of this review. The first group are those which are a function of the mode of compaction, which are discussed in Section 2.4.2, and the second are a function of the material, and is discussed in Section 2.4.3.

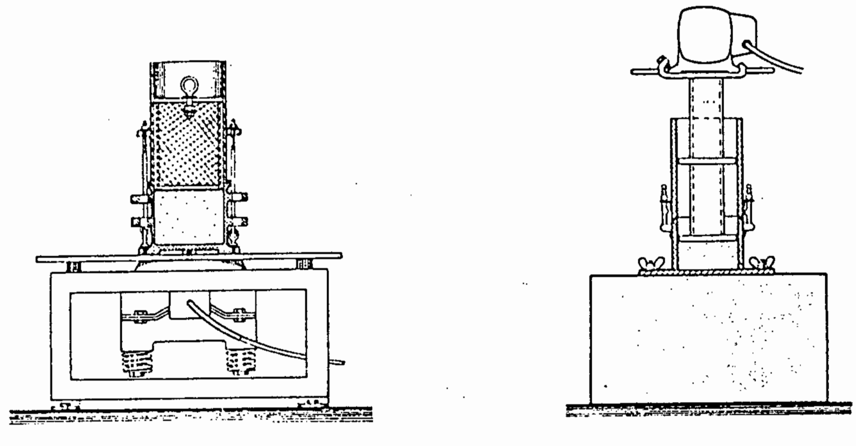


FIGURE 2.3:- SCHEMATIC REPRESENTATION OF VIBRATORY TESTING APPARATUS. (AFTER FORSSBLAD, 1981)

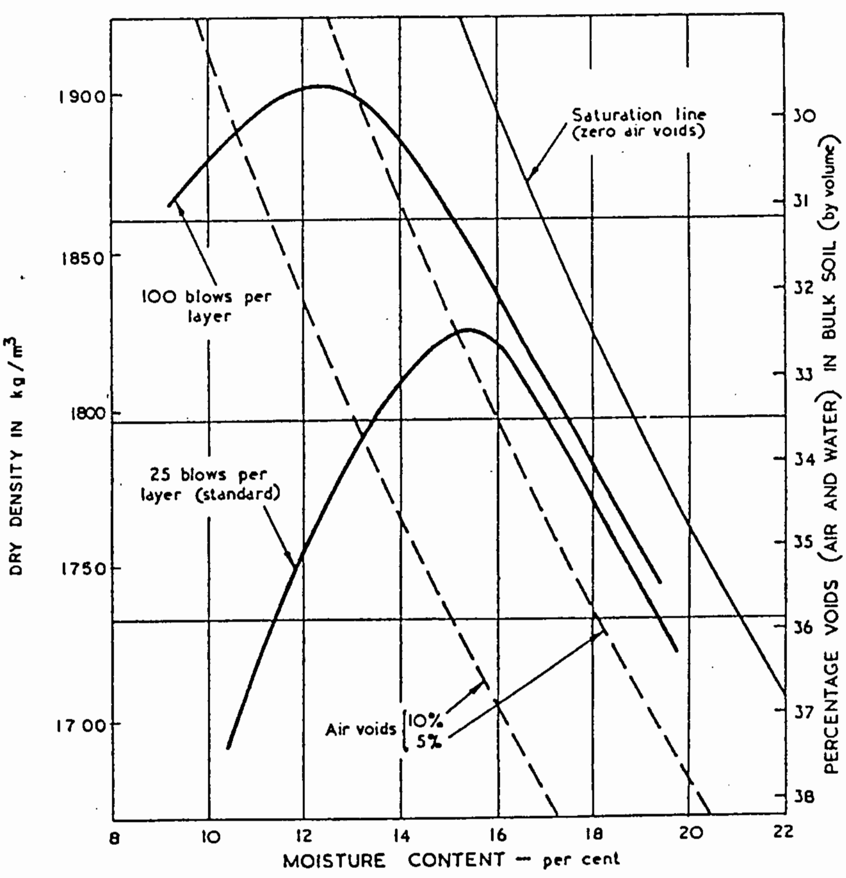


FIGURE 2.4:- EFFECT OF DIFFERENT AMOUNTS OF COMPACTION ON DRY DENSITY OF SANDY CLAY SOIL.
(AFTER SOIL MECHANICS FOR ROAD ENGINEERS, 1968)

2.4.2 Method of compaction.

The method of laboratory compaction affects the density achieved. When the same soil-type is subjected to two impact tests in which the only difference is the height from which the mass is dropped, the resulting densities will differ (cf Figure 2.4). If moreover a third similar sample is tested using a vibratory method the result can be expected to differ again. In a Proctor-type test the factors which are relevant to the compactive effort per unit volume are:

* size and shape of mould
* type and dimensions of the rammer and rammer guide
* weight, velocity, energy and momentum of the rammer
* percent of total compaction energy applied in each tamp (Johnson and Sallberg, 1962)

For a vibratory test method the following aspects of the test method have been found to have an affect:

* mode of vibration i.e. Whether the vibration is imparted by a vibratory table (electromagnetic or cam-driven), a hand-held tamper or a device clipped to the side of the mould.
* frequency of vibration
* amplitude of vibration
* mould size and shape
* surcharge pressure and whether it is applied as a single dead weight, a series of loose plates or by a spring
* time of vibration
* number of layers in the total sample.

In both impact and vibratory compaction the effects of the above factors interact to produce a specific dry density and OMC for a given soil. When quoting maximum dry density and OMC for a soil it is therefore necessary to state explicity the test conditions of which these parameters are a product.

The effect of the factors affecting laboratory vibratory compaction are discussed in greater detail in Section 2.5.

2.4.3 Material factors.

The differing responses to compactive effort in both the field and the laboratory of cohesionless and cohesive materials was the main impetus behind developing a laboratory vibratory test for cohesionless material. This implies that the characteristics of a soil influence its compactibility. The latest ASTM standard laboratory test using vibration ASTM D 4253-83 and its predecessor ASTM D 2049-69 are intended for use with cohesionless, free-draining soil only. That they are not intended to apply to materials exhibiting some cohesive properties seems to stem more from the fact that impact tests have been found adequate for these materials, rather than because they cannot be compacted by vibration. In the field vibration is used to compact rock fill, soil cement, sand, crushed stone, silt and clay. (Forssblad 1981) Even such materials as asphalt are compacted with vibratory equipment (Forssblad 1977).

Within in the range of soils which are compacted in the field many materials exist which are neither pure clay, silt or uniformly graded sand. Graded crushed stone which is a composite of many size particles is particularly difficult to compact. A graded crushed stone of the type used for basecourse construction may contain particles smaller than 0.075mm and as large as 75mm. The fines in the mix may exhibit a degree of plasticity. In some cases the amount of plastic fines in a mix may be too small to result in the whole mix having cohesive properties, while if larger proportions are present the plastic fines may dominate the mix. Thus a soil may exhibit a range of characteristics which depending on the dominance of the constituents, will influence compaction by vibration to a greater or lesser degree.

Characteristics of a soil which may affect compaction include:

* grading
* cohesion
* plasticity of fines
* permeability
* moisture content
* particle shape and strength.

These characteristics are in fact those by which soils are classified by such systems as the Unified Soil Classification System. Some aspects of the influence of each of the above factors on compaction is considered in the following paragraphs. (As the same method of laboratory compaction was not used by all the researchers, the trends indicated should be viewed qualitatively only).

2.4.3.1 Grading

Since the grading of a soil is one of the major determinants in classifying a soil, the importance of grading is self-evident. It is well-known for example that sand requires a different approach to compaction than clay.

However even much smaller differences in grading, such as those discriminating between a uniformly graded sand and a well graded sand, can have a significant bearing on the density achieved with a particular compaction test. (cf Fig 2.7).

The concept "grading" is sometimes understood to refer to "particle size distribution" only, but there are a number of other aspects, which are also part of grading which may be identified, and which jointly and severally affect compaction.

These include particle size distribution, maximum particle size, percentage of fines and the ratio of coarse to fine material. Several researchers have in the past tried in various ways to relate each of these factors to the density achieved. For soils with uniform particle sizes it has been found easier to determine relationships than, for example, well-graded granular materials like basecourse quality crushed stone, which can contain particle sizes from 37.5mm down to smaller than 0.075mm.

The Talbot formula (cf Appendix A.1) and the Uniformity Coefficient C_u are two methods commonly used for quantifying the particle size distribution.

Gradings which fit the Talbot equation are "well-graded" and the formula is often used to specify particle size distribution for crushed stone basecourse. The uniformity coefficient (C_u) gives an indication of the spread of particle sizes. The naming of this coefficient is unfortunate since well-graded materials have a high coefficient of uniformity, while single size materials have a low one.

Figure 2.5 illustrates how, on the basis of experiments with a vibratory method, Johnston, (1972) derived a relationship between C_u and dry density. The material used in the experiments comprised a subangular to rounded grain with all material retained on the 0.075mm sieve.

Researchers such as Maddison (1944), Cumberledge and Cominsky (1972) and Turnbull and Foster (1957) have found that there is an optimum ratio of coarse to fine aggregate which gives the maximum density with a given test method. Work by Maddison is illustrated graphically in Figure 2.8.

Maddison mixed single-size aggregates of hard crushed rock of three sizes (0.5 to 12.7, 12.7 to 19.0 and 19 to 25mm) to a silty clay (sand, silt and clay contents of 58, 18 and 24 percent, respectively LL = 26, PI = 5). It was found that for the compacted mixtures with up to 25% of any of the single-sized aggregates, the coarse aggregate merely "floated" in the finer material. With higher coarse aggregate contents, the dry unit weight of the mixture increased up to a coarse aggregate content of about 50%. With coarse aggregate contents of more than about 70% the dry density dropped once more due to lack of fines. The above effects are illustrated pictorially in Figure 2.6.

Work by Cumberledge and Cominsky is shown in Figure 2.9A and 2.9B. These researchers found an optimum percentage of plus 4.75mm material to achieve maximum dry density. It is interesting to note that the optimum was not the same for each of the methods of test used. (cf Fig 2.9).

Work by Turnbull and Foster is illustrated in Figure 2.10. The samples tested were graded crushed limestone suitable for base course of which some had maximum size aggregate of 19mm and others 37.5mm. The results show that at low compactive effort, the 37.5mm maximum size aggregate resulted in a markedly higher maximum unit weight. At compactive efforts of the order of Modified AASHTO, the effect of the maximum size of particle was less pronounced.

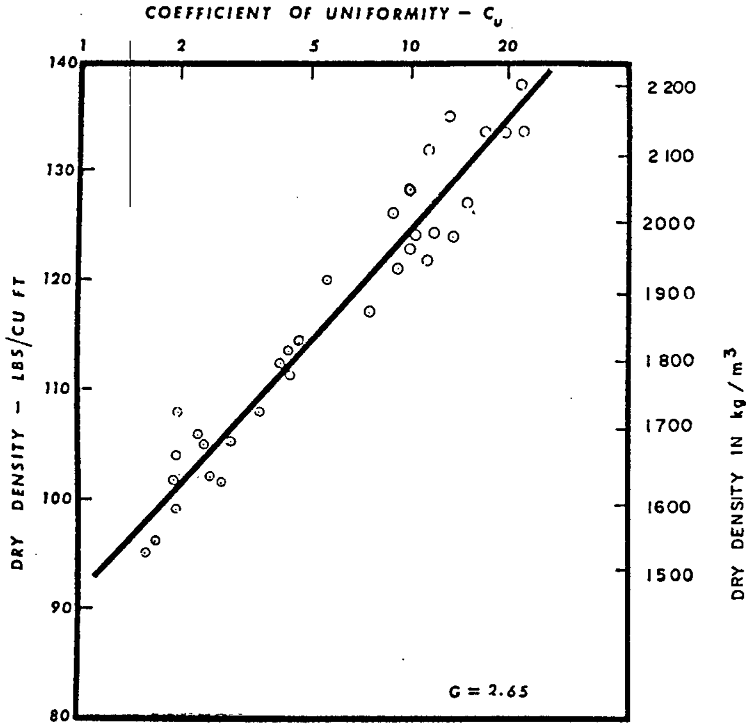
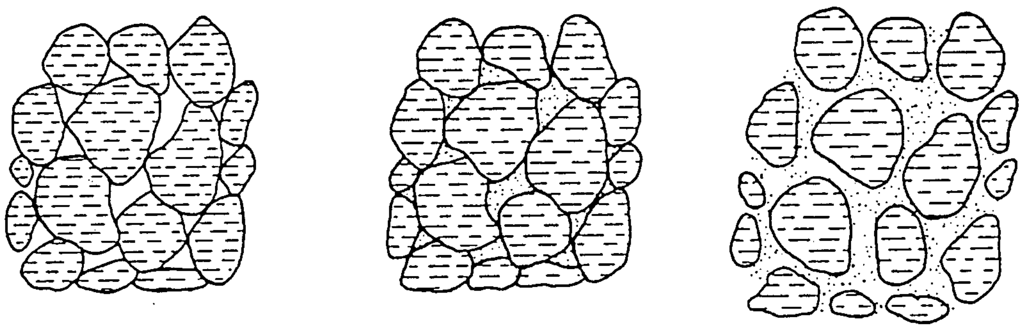


FIGURE 2,5:– EMPIRICAL RELATIONSHIP BETWEEN MAXIMUM DENSITY AND COEFFICIENT OF UNIFORMITY (AFTER JOHNSTON,1972)



a) LACK OF FINES

b) OPTIMUM PROPORTIONS OF COARSE AND FINE

c) COARSE AGGREGATE FLOATING IN FINES

FIGURE 2.6 :–INFLUENCE OF FINES ON DENSITY

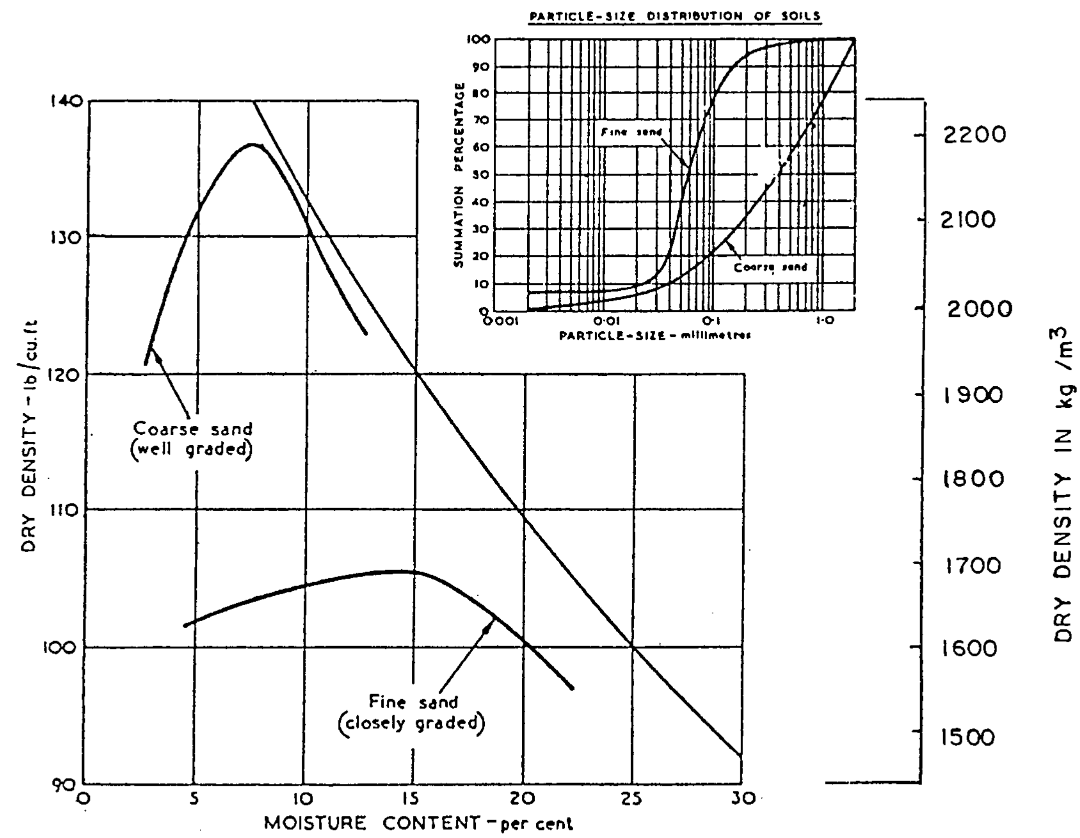


FIGURE 2.7:- DRY DENSITY/MOISTURE CONTENT CURVES FOR TWO SANDS WITH DIFFERENT PARTICLE-SIZE DISTRIBUTIONS. (AFTER SOIL MECHANICS FOR ROAD ENGINEERS, 1968)

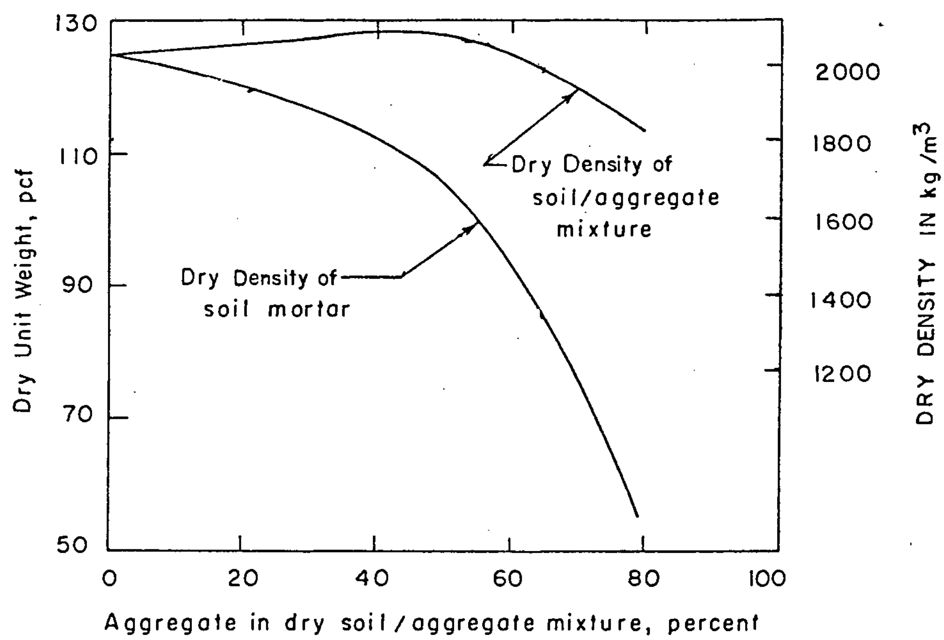


FIGURE 2.8 :- COMPACTION OF SOIL MORTAR AT OPTIMUM MOISTURE CONTENT WITH DIFFERENT PERCENTAGES OF AGGREGATE. (AFTER JOHNSON AND SALLBERG, 1962)

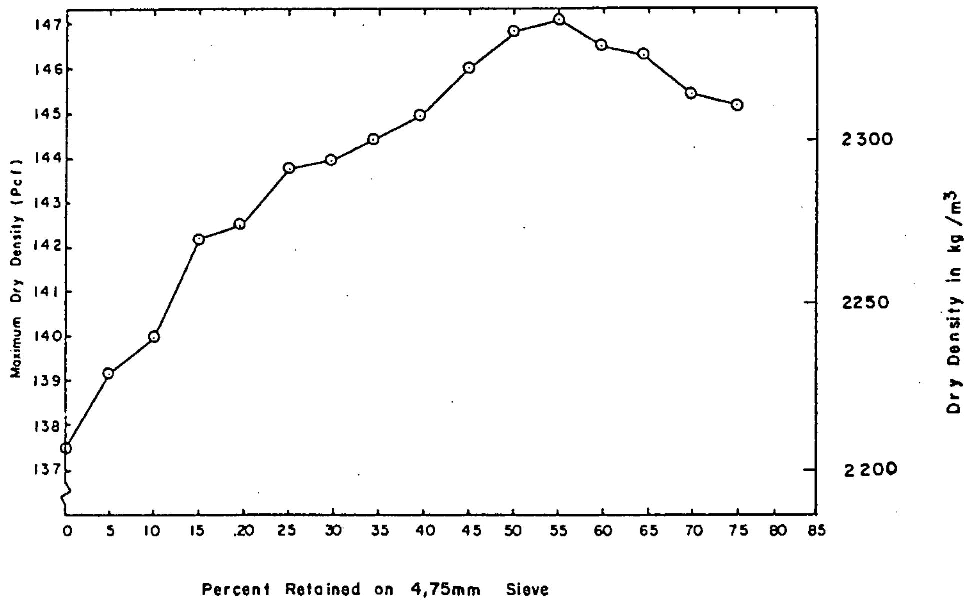


FIGURE 2.9A :- MAXIMUM DRY DENSITY OBTAINED BY STANDARD AASHO METHOD (152mm MOULD) BY VARYING THE PERCENTAGES OF MATERIAL RETAINED ON THE 4,75mm SIEVE

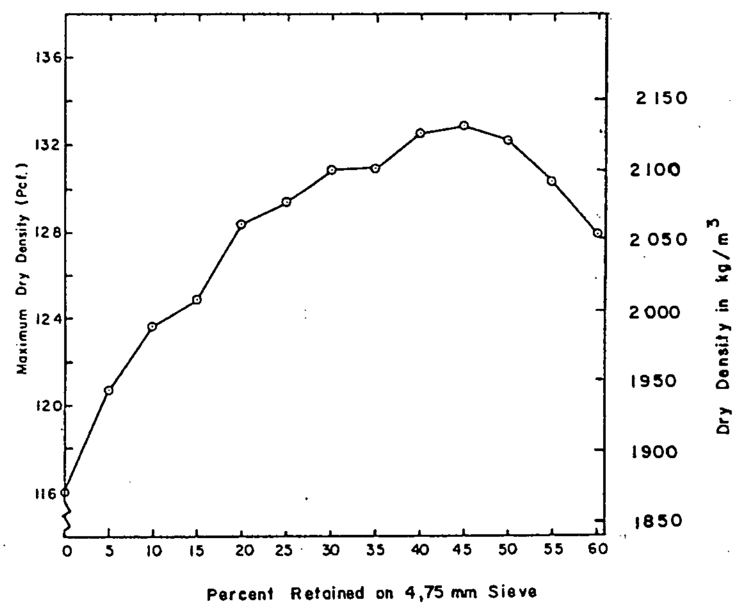


FIGURE 2.9B :- MAXIMUM DRY DENSITY OBTAINED BY VIBRATORY METHOD IN 152mm MOULD BY VARYING THE PERCENTAGES OF MATERIAL RETAINED ON THE 4.75mm SIEVE.

(AFTER CUMBERLEDGE AND COMINSKY, 1972)

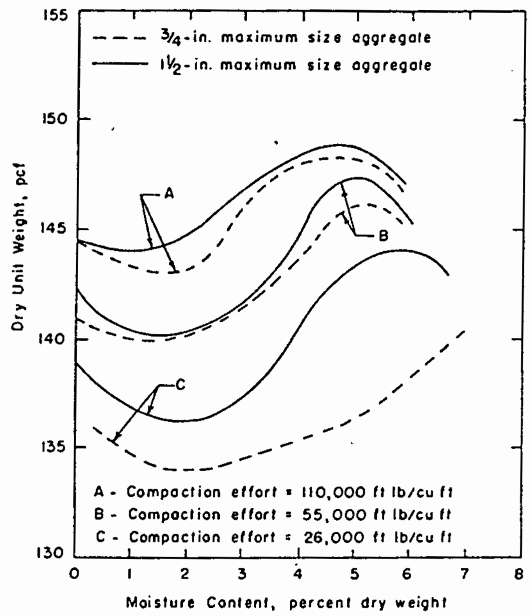


FIGURE 2.10 :- COMPARISON OF RESULTS OF LABORATORY COMPACTION OF GRADED CRUSHED LIME-STONE WITH MAXIMUM SIZE OF 3/4 IN. AND 1 1/2 IN. UNDER THREE COMPACTION EFFORTS.

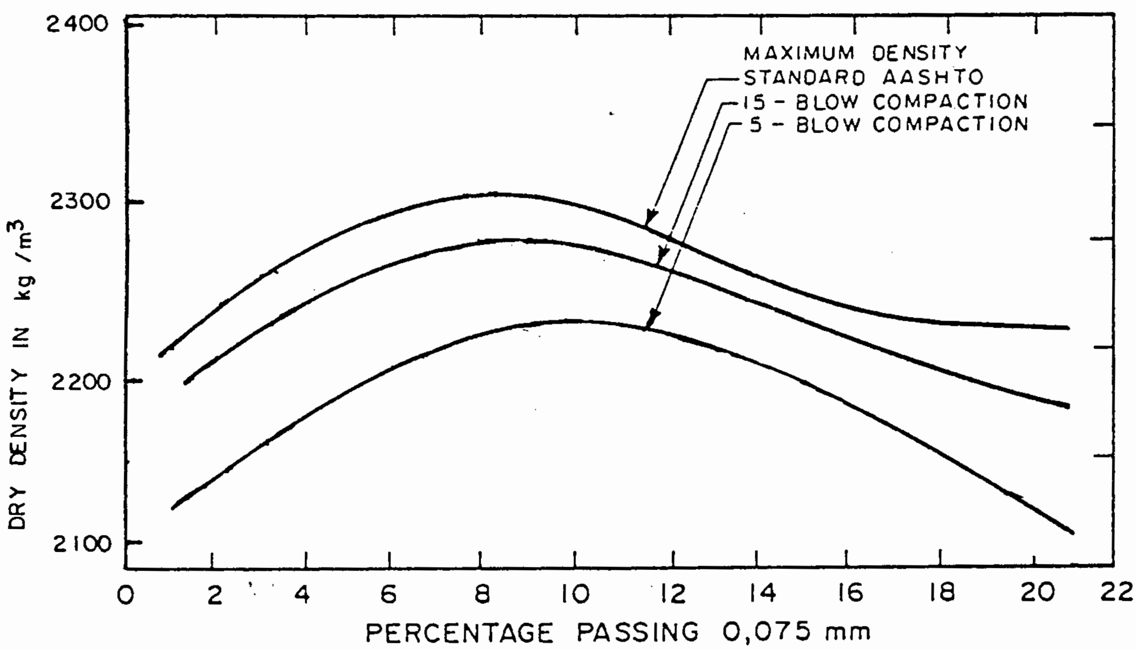


FIGURE 2.11:- INFLUENCE OF PERCENTAGE FINES ON DENSITY (AFTER YODER AND WITCZAK, 1975)

Yoder and Witczak (1975), carried out tests on soils containing varying percentages of fines. The results, showed that for a given compactive effort there existed an optimum percentage of minus 0.075mm material. (cf Fig 2.11). The effect of plasticity of fines is discussed separately under Section 2.4.3.3.

From the above it may be concluded that the grading of a soil, especially a graded crushed stone has a significant effect on the dry density achieved with a given test method.

2.4.3.2 Particle shape and strength.

There are a number of ways in which the geometric characteristics of coarse aggregates may be quantified. These include angularity number, particle index, coefficient of angularity and specific rugosity (cf List of symbols).

Van der Merwe (1984) showed that a good relationship existed between "specific rugosity" and the dry density which could be achieved with a specific laboratory vibratory compaction technique. Angular aggregates with high specific rugosity and high macro surface voids gave lower densities. Figure 2.12 shows the relationship between density and specific rugosity for a number of graded crushed stone materials which fitted the Talbot equation.

Holubec and D'Appollonia (1972) reported that the maximum density obtainable at a given compactive effort decreased as the angularity of the particles increased. Roston et al (1976) suggest however that angular materials are merely more difficult to compact, but that with vibratory rather than impact type testing good results can be achieved. This would indicate that whereas angularity may inhibit compaction by an impact method it may not be that significant if vibration is used.

So long as the individual particles do not break down during the compaction process the density achieved is independent of the strength of the particles. The degradation of the larger particles of graded crushed stone during compaction tests, particularly the impact tests, is well known. When the aggregate

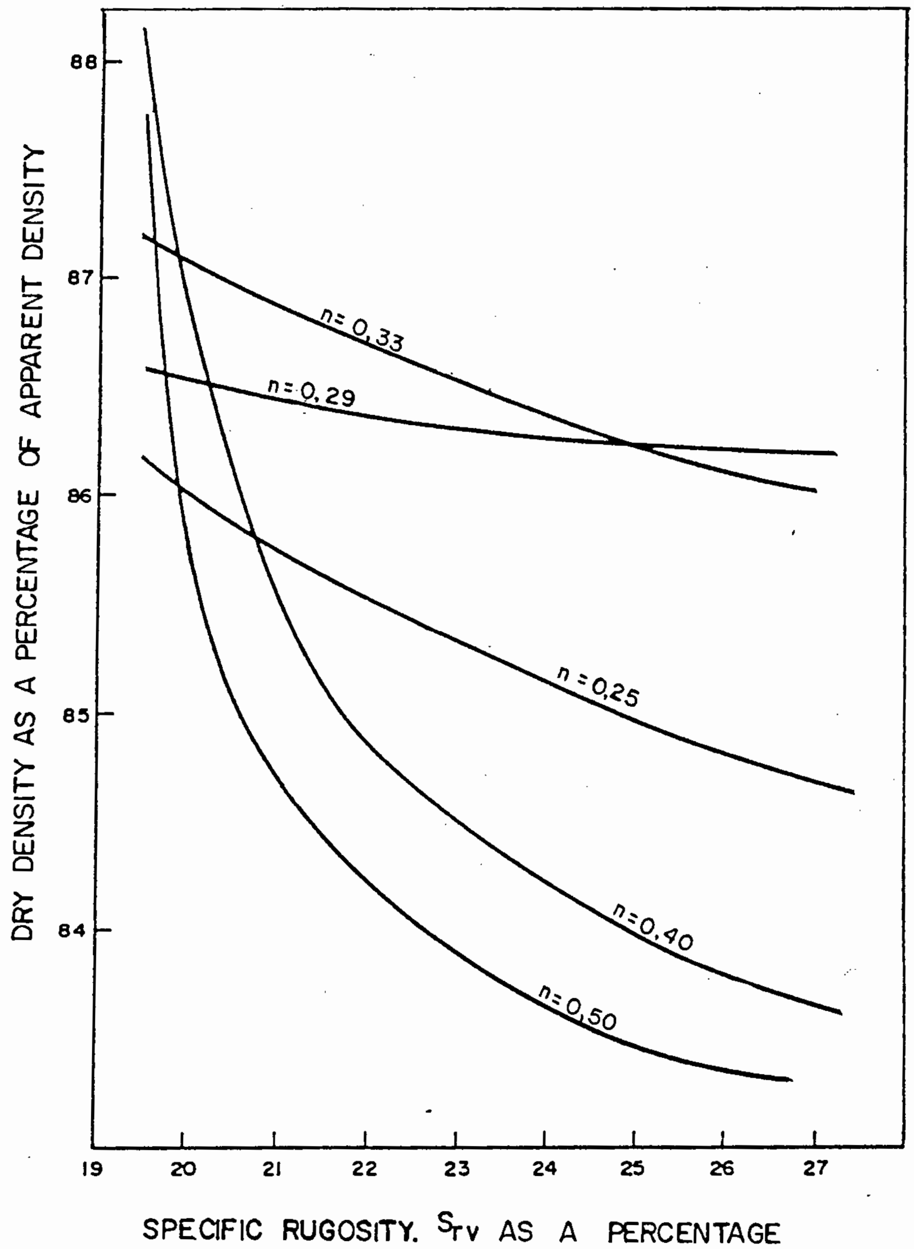


FIGURE 2.12:- RELATIONSHIP BETWEEN DENSITY, GRADING AND GEOMETRIC CHARACTERISTICS OF AGGREGATES (AFTER van der MERWE, 1984)

is broken down during compaction, the grading is affected and consequently the density which can be achieved is also modified (cf Section 2.4.3.1). Pettibone and Hardin (1965), van der Merwe (1984) and Cumberledge and Cominsky (1972) reported that for crushed stone significantly less degradation took place under laboratory vibratory compaction than under impact tests. Also more breakdown occurred when compacting in the dry than when substantial amounts of moisture were present.

2.4.3.3 Plasticity of fines.

The plasticity of a soil is defined by the plasticity index (PI) which represents the range of moisture contents over which the plastic properties dominate soil behaviour. The plasticity index is defined as the difference between the liquid limit and the plastic limit. (i.e. PI = LL - PL). These limits which are moisture contents are measured by means of standard tests. The tests are normally carried out on that part of the soil passing a 0.425mm sieve. This means that for a graded crushed stone for example, the plasticity is a property of the fine fraction but not of the soil as a whole. The extent to which the plasticity of fines influences the soil properties therefore depends on the percentage of plastic fines in the mix.

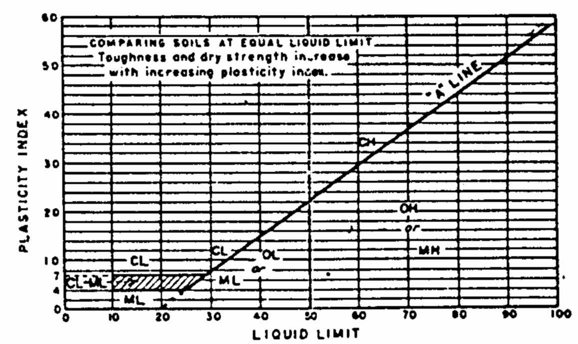
The plasticity index of a soil is one of those properties which to a large extent affects its classification according to the Unified Soil Classification System. To read the plasticity chart developed by Casagrande, shown in Figure 2.13, it is necessary to plot a point which has as co-ordinates the LL and the PI. The soil can then be classified by observing the position of the point relative to the A-line. The A-line was empirically determined after extensive testing on different soil types. The equation of the A-line is : PI = 0.73 (LL - 20). Soils which fall above the line are classed as inorganic clays and those which fall below, as organic silts and clays. If however the LL < 25 there is a considerable amount of overlapping as indicated by the shaded area.

UNIFIED SOIL CLASSIFICATION
INCLUDING IDENTIFICATION AND DESCRIPTION

FIELD IDENTIFICATION PROCEDURES (Excluding particles larger than 3 inches and basing fractions on estimated weights)				GROUP SYMBOLS ‡	TYPICAL NAMES	INFORMATION REQUIRED FOR DESCRIBING SOILS	LABORATORY CLASSIFICATION CRITERIA
COARSE GRAINED SOILS — More than half of material is larger than No. 200 sieve size ‡ (The No. 200 sieve size is about the smallest particle visible to the naked eye)	GRAVELS — More than half of coarse fraction is larger than No. 4 sieve size. (For visual classifications, the ¼" size may be used as equivalent to the No. 4 sieve size.)	CLEAN GRAVELS (Little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes	GW	Well graded gravels, gravel-sand mixtures, little or no fines.	Give typical name; indicate approximate percentages of sand and gravel, max. size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbol in parentheses.	$C_u = \frac{D_{60}}{D_{10}}$ Greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between one and 3
			Predominantly one size or a range of sizes with some intermediate sizes missing.	GP	Poorly graded gravels, gravel-sand mixtures, little or no fines.		Not meeting all gradation requirements for GW
		GRAVELS WITH FINES (Appreciable amount of fines)	Non-plastic fines (for identification procedures see ML below).	GM	Silty gravels, poorly graded gravel-sand-silt mixtures.		Atterberg limits below "A" line, or PI less than 4 — Above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols.
			Plastic fines (for identification procedures see CL below).	GC	Clayey gravels, poorly graded gravel-sand-clay mixtures.	For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics.	Atterberg limits above "A" line with PI greater than 7
	SANDS — More than half of coarse fraction is smaller than No. 4 sieve size.	CLEAN SANDS (Little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes.	SW	Well graded sands, gravelly sands; little or no fines.		$C_u = \frac{D_{60}}{D_{10}}$ Greater than 6; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between one and 3
			Predominantly one size or a range of sizes with some intermediate sizes missing.	SP	Poorly graded sands, gravelly sands; little or no fines.	EXAMPLE:- Silty sand, gravelly; about 20% hard, angular gravel particles ½-in. maximum size; rounded and subangular sand grains coarse to fine; about 15% non-plastic fines with low dry strength; well compacted and moist in place; alluvial sand; (SM)	Not meeting all gradation requirements for SW
		SANDS WITH FINES (Appreciable amount of fines)	Non-plastic fines (for identification procedures see ML below).	SM	Silty sands, poorly graded sand-silt mixtures.		Atterberg limits below "A" line or PI less than 4 — Above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols.
			Plastic fines (for identification procedures see CL below).	SC	Clayey sands, poorly graded sand-clay mixtures.		Atterberg limits above "A" line with PI greater than 7

Laboratory classification criteria (coarse grained soils): Use grain size curve in identifying the fractions as given under field identification. Determine percentages of gravel and sand from grain size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size) coarse grained soils are classified as follows:- Less than 5% GW, GP, SW, SP. More than 12% GM, GC, SM, SC. 5% to 12% Borderline cases requiring use of dual symbols.

FINE GRAINED SOILS — More than half of material is smaller than No. 200 sieve size. (The No. 200 sieve size is about the smallest particle visible to the naked eye)	IDENTIFICATION PROCEDURES ON FRACTION SMALLER THAN No. 40 SIEVE SIZE: DRY STRENGTH (CRUSHING CHARACTERISTICS)	DILATANCY (REACTION TO SHAKING)	TOUGHNESS (CONSISTENCY NEAR PLASTIC LIMIT)	GROUP SYMBOLS	TYPICAL NAMES	INFORMATION REQUIRED FOR DESCRIBING SOILS
SILTS AND CLAYS — Liquid limit less than 50	None to slight	Quick to slow	None	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity.	Give typical name; indicate degree and character of plasticity, amount and maximum size of coarse grains; color in wet condition, odor if any, local or geologic name, and other pertinent descriptive information; and symbol in parentheses.
	Medium to high	None to very slow	Medium	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.	
	Slight to medium	Slow	Slight	OL	Organic silts and organic silt-clays of low plasticity.	For undisturbed soils add information on structure, stratification, consistency in undisturbed and remolded states, moisture and drainage conditions.
SILTS AND CLAYS — Liquid limit greater than 50	Slight to medium	Slow to none	Slight to medium	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.	
	High to very high	None	High	CH	Inorganic clays of high plasticity, fat clays.	EXAMPLE:- Clayey silt, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (ML)
	Medium to high	None to very slow	Slight to medium	OH	Organic clays of medium to high plasticity.	
HIGHLY ORGANIC SOILS	Readily identified by color, odor, spongy feel and frequently by fibrous texture.			Pt	Peat and other highly organic soils.	



PLASTICITY CHART
FOR LABORATORY CLASSIFICATION OF FINE GRAINED SOILS

‡ Boundary classifications:- Soils possessing characteristics of two groups are designated by combinations of group symbols. For example GW-GC, well graded gravel-sand mixture with clay binder.
‡ All sieve sizes on this chart are U.S. standard.

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FIGURE 2,13 :— UNIFIED SOIL CLASSIFICATION CHART.
From drawing 103-D-347.
(EARTH MANUAL, 1974)

In general the finer the soil the greater the PI, and for the same liquid limit the greater the PI the greater the cohesion (i.e. strength) of the soil.

As mentioned in Section 2.4.3 the influence of plastic fines on the compaction of a graded crushed stone depends on the relative proportion of plastic fines to the whole. Holtz and Lowitz (1957) conducted an extensive series of tests to determine the compaction characteristics of granular soils. The tests were conducted on a large range of gradations as shown in Figure 2.14. A non-standard impact-type test was used. Some of the results are presented in Figure 2.15 a-c. It can be seen in this figure that for soils with varying percentages of plastic fines the optimum grading for maximum density is dependent on the PI.

Although the standard laboratory vibratory tests, such as ASTM D 2049-69 set cohesionless and free-draining as prerequisite properties of soil to which the test is applied, these vibratory tests have been used reportedly with success on soils with some plasticity (Holtz, 1972).

An interesting study was conducted by Townsend (1972), who compared the maximum dry density from a vibratory method with Standard AASHTO maximum dry density for sands with varying amounts of fines. Measured percentages of low and medium plasticity fines (PI 2 to 10) were added to poorly graded and nearly well graded sand. The results indicated that a greater percentage of fines could be accommodated by a uniform sand and that uniform sand densified by vibration more effectively than a well-graded sand. Moisture and plasticity were found to be interrelated factors which greatly affected compaction. Saturation facilitated vibratory compaction of the low plasticity mixtures, but for more plastic mixtures, adhesion of the fines to the sand grains restricted vibratory densifications. The plasticity of the fines when compacted in the oven-dry state therefore has very little influence on compaction.

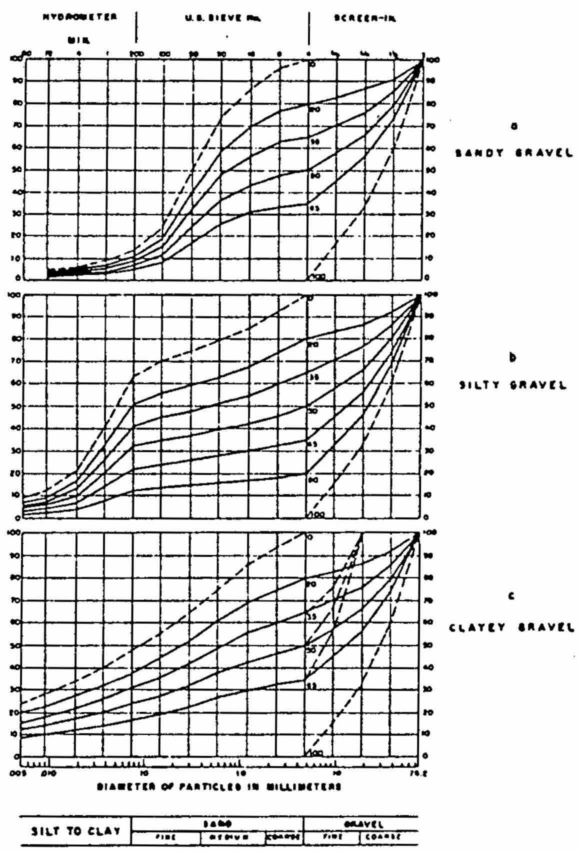
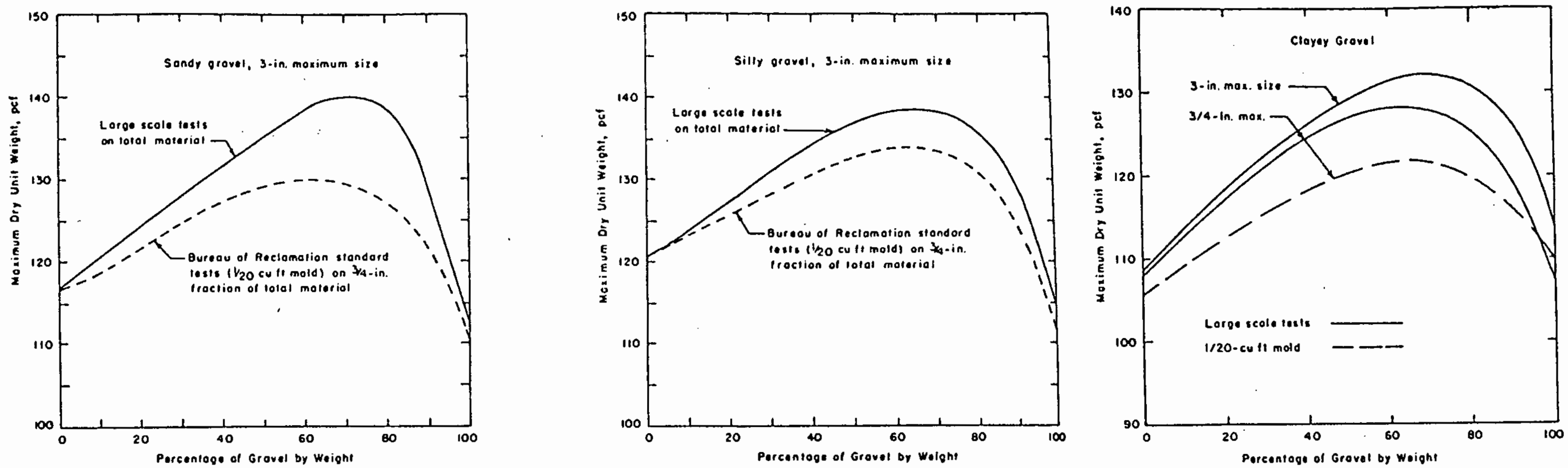


FIGURE 2.14 :- GRADATION OF THE NUMEROUS MIXTURES USED IN INVESTIGATION OF COMPACTION CHARACTERISTICS OF GRAVELLY SOILS (HOLTZ AND LOWITZ, 1957)



a) FINE SOIL FRACTION IS NON PLASTIC SANDY SOIL OF SW—SM GROUP

b) FINE SOIL FRACTION IS SILTY SOIL OF ML GROUP WITH LL = 26, PI = 4

c) FINE SOIL FRACTION IS CLAY OF CL-CH GROUP WITH LL=49, PI = 28

FIGURE 2.15:-RELATIONSHIP BETWEEN MAXIMUM DRY DENSITY AND GRAVEL CONTENT. (AFTER HOLTZ AND LOWITZ, 1957)

2.4.3.4 Moisture content and permeability.

All soils compact better and reach higher densities at some moisture contents than at others. The moisture content at which the maximum dry density is achieved with a given compactive effort is known as the optimum moisture content (OMC). The OMC is dependent on the method of compaction (cf 2.4.1). It also depends on the material and is generally higher for fine grained, than for coarse grained soils. Figure 2.16 shows moisture content-dry density relationships for various types of soils.

If one compares the OMC and maximum dry density for two samples of the same material, one compacted with Standard and the other with Modified AASHTO compactive effort, the maximum dry density will be 5 - 10% higher with the latter method. The difference is about 5% for granular materials and 10% for cohesive soils. The optimum moisture content is usually some 3 - 8% lower for the higher (Modified AASHTO) compactive effort. As the method used in the laboratory affects the maximum dry density and OMC, it follows, that with a different method of compaction in the field, the OMC and dry density will also be different (cf Fig 2.4).

Consider the typical moisture content-dry density relationship of a silt or clay. At the lower water contents the internal friction and adhesion between particles contributes to the resistance to compaction. At higher water contents the material is easier to compact and an optimum water content exists where the maximum dry density is obtained. Above the optimum moisture content the soil density is reduced, because the water is held by capillary forces preventing particles from rearranging themselves into a denser packing.

For more free-draining soils such as sands and crushed stone, with less than 5% fines, the water is pressed out when the particles relocate and the OMC normally corresponds to that state in which the uncompacted soil is (cf Appendix A.7, saturation).

For cohesionless free-draining sand, two moisture contents normally exist at which the maximum dry density can be achieved (cf Fig 2.16). This is either when the sample is initially saturated or when totally dry. When sand is moist(i.e. the voids are partially filled with water) the apparent cohesion from the resulting capillary forces inhibits compaction.

Graded crushed stone is one of the most difficult materials to compact as the moisture-density relationship may have a number of peaks. Provided the material is free-draining, saturating it prior to compaction leads to the highest dry density (van der Merwe, 1984). Lee, (1972), van der Merwe (1984) and Forssblad (1981) refer to "one-an-a-half-peak" relationships such as shown in Figure 2.17 for this type of material.

Graded crushed stone has been found to densify better under vibration than under impact (van der Merwe (1984), Felt (1958)). Although standard vibratory compaction tests such ASTM D 2049-69 have been used with success on soils as PI's with high as 12 (Holtz, (1972)), it has been shown that for an increase in plasticity index for the same liquid limit, the permeability decreases rapidly (Nettles and Calhoun, 1967). Despite this the test method stresses that the material must be free-draining.

Nettles and Calhoun (1967) determined the permeability of crushed stone at various compacted densities. Four gradings were tested (maximum stone size 19mm and 25mm) and the percentage fines was varied between 0.5 and 10%. All the gradings fitted the Talbot equation (cf Appendix A.1).

For all the gradings it was found that permeability decreased considerably with increasing density. It was concluded that at 100% Mod AASHTO, the material with 0% fines was highly permeable, with 5% fines it was marginally permeable and with 10% fines, impermeable.

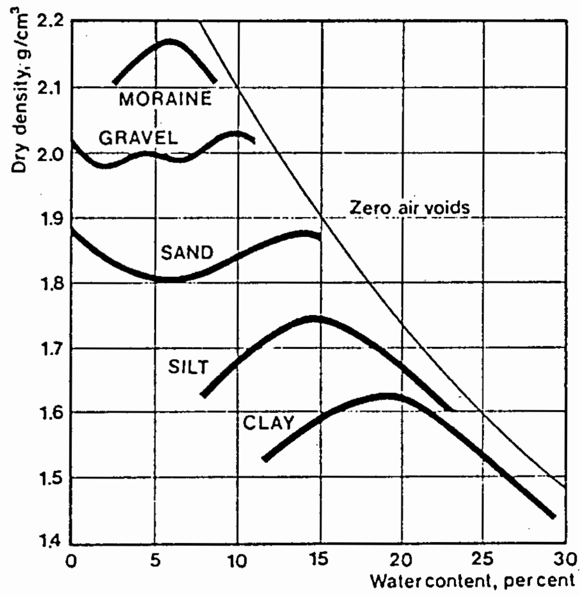


FIGURE 2.16:- LABORATORY COMPACTION CURVES FOR DIFFERENT TYPES OF SOILS. (FORSSBLAD, 1981)

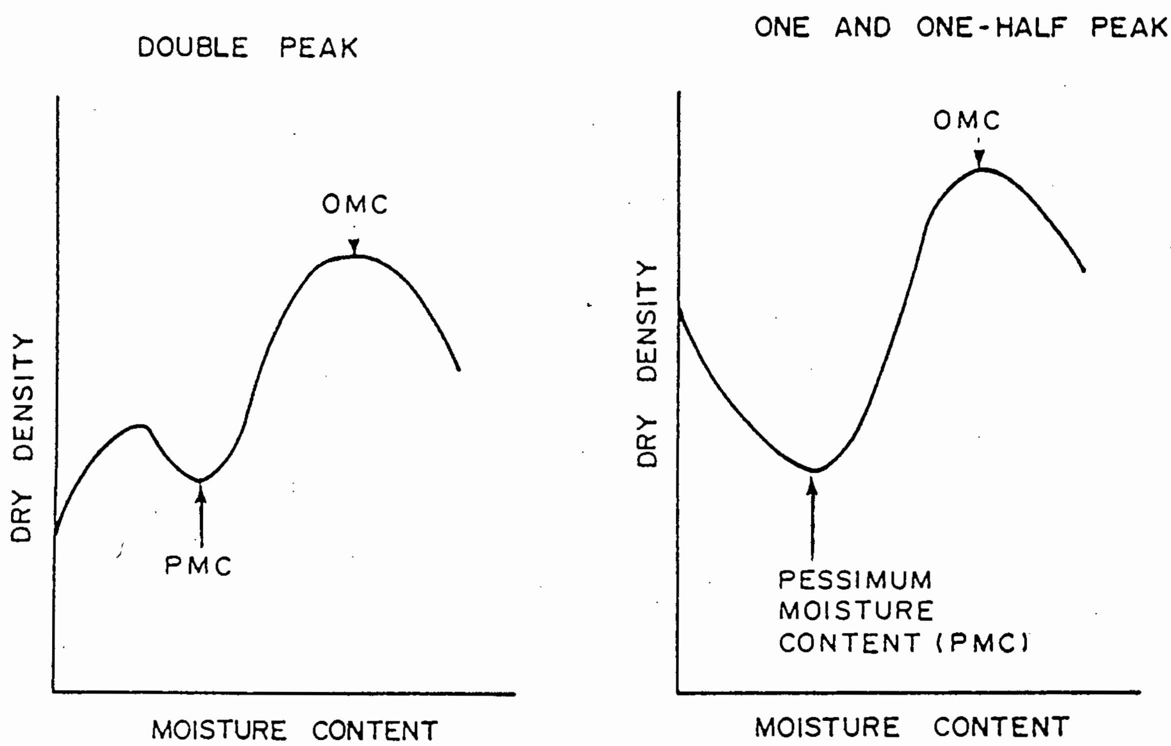


FIGURE 2.17:-TYPES OF MOISTURE CONTENT / DRY DENSITY CURVES (AFTER LEE AND SUEDKAMP, 1972)

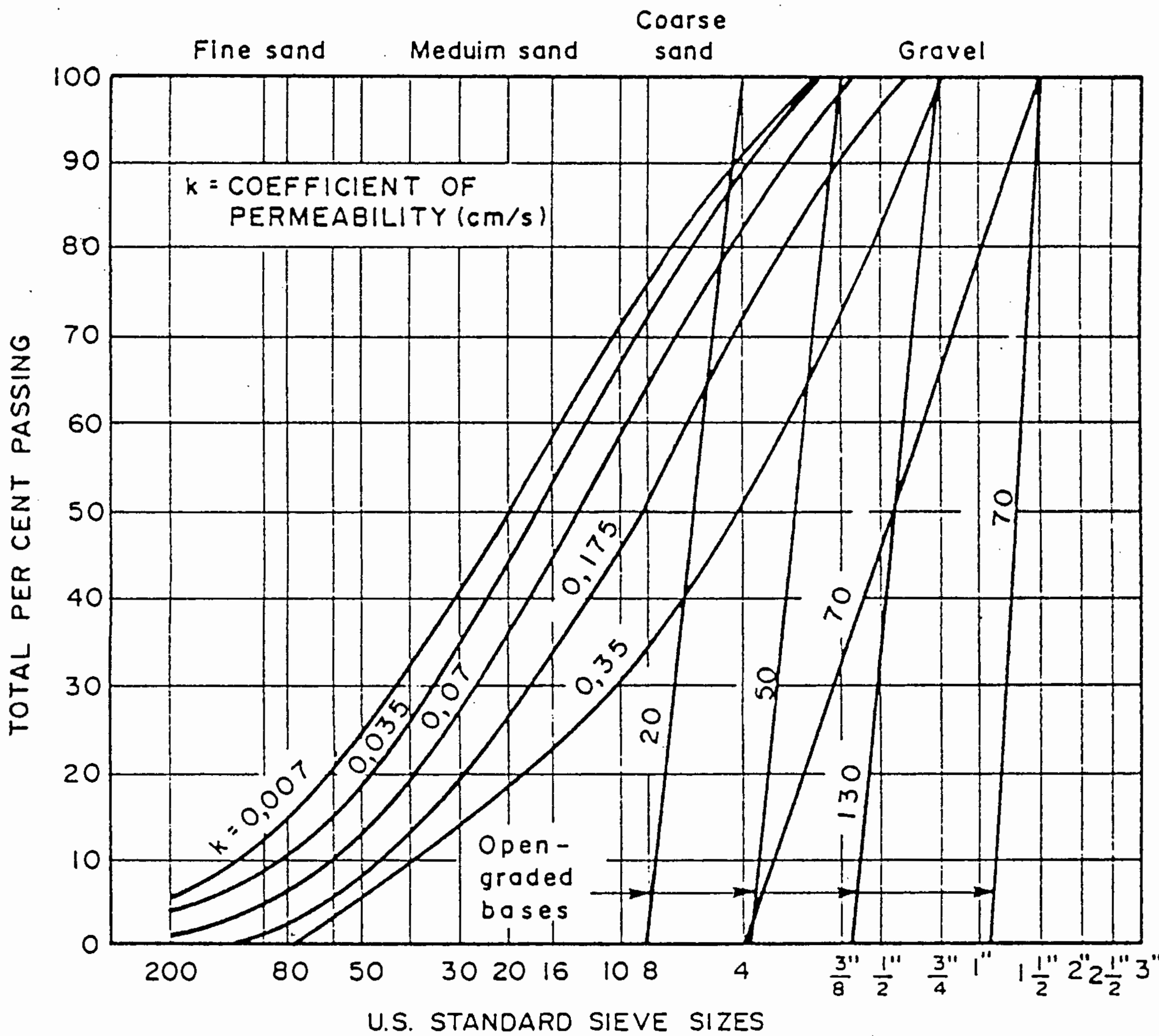


FIGURE 2.18:—THE INFLUENCE OF PARTICLE SIZE DISTRIBUTION ON PERMEABILITY (after Cedegren, 1972)

2.5 The mechanism of laboratory vibratory compaction for cohesionless material.

In section 2.2 the historical development of laboratory vibratory compaction was discussed, and the close link between vibratory compaction and the concept of relative density was highlighted. In section 2.3 it is shown that the soil types to which the standard vibratory tests such as ASTM Designation: D2049-69 and ASTM Designation: D4253-83 apply, are required to be cohesionless and free-draining. In section 2.4 attention was drawn to the factors which influence laboratory compaction. In particular it was stressed that both the method of compaction and the material have a significant influence on the maximum dry density and OMC achieved.

The parameters which affect laboratory compaction interact to produce a unique result for a given compaction method and material. When studying the mechanism of vibratory compaction in the laboratory therefore it is important to specify the full test method used. For this reason the major research projects in the United States which formed the basis for ASTM: D2049-69 and ASTM: D4253-83 are studied as separate units in sections 2.5.1 to 2.5.10. Moreover, as the dry densities achieved are generally only of the order of Standard AASHTO dry density, it should be borne in mind that the procedures are not necessarily optimal. Nevertheless, some clear trends are evident.

The fact that most of the available papers concerned developments in the USA, where a preference for a vibratory table method was shown at an early stage, led the research in that country in a specific direction. The European practice, where the mould is firmly attached to a base and vibration is imparted from the top by a vibratory rammer, is less refined at this stage. This kind of method may however prove to be a most promising avenue for further research.

The research projects in the USA are dealt with in chronological order, so that the sequence in which developments took place is evident.

2.5.1 Earth Manual - Test E12 (1951).

The first standard test for determining a maximum dry density for use in the relative density formula was published in the (tentative) first edition of the US Bureau of Reclamation's Earth Manual in 1951. The method was based on vibration and applied to cohesionless, free-draining soils. The following features of the test are significant:

i) The size of mould in which material was compacted depended on the maximum size soil particle. For maximum size particles of 3, 3/4 and 3/8 inches (76, 19 and 9.5 mm) mould sizes were 1.0; 0.5 and 0.1 ft^3 respectively (0.0283; 0.0142; 0.0028 m^3). The smallest of these had a diameter of 6 in. (152mm) and was similar to the standard mould for CBR. The larger moulds had diameters of 11 and 13 in (297 and 330mm).

ii) The soil samples were tested under initially saturated conditions. (cf Appendix A.7)

iii) The saturated soil was added to the measure with the vibratory device in operation.

iv) A minimum vibration time of 1 minute was set

v) No surcharge weight was involved.

vi) The vibratory device was specified as being of the foundry type which was clipped onto the side of the mould, and neither frequency nor amplitude of vibration were specified.

vii) Soils were generally classified as cohesionless and free-draining for the purposes of the test if the percentage passing the 0.075mm sieve was less than 12%. (cf Section 2.3)

The method was found to be unsatisfactory in many respects and in 1954 Section D of Sub-committee R-3 of the ASTM Committee D18 was commissioned to develop methods for determining, amongst other things, the maximum density of granular soils for use in the relative density specifications. The committee stated in their terms of reference that:

"The consideration of a test will include a study of the characteristics of the material on which it may be used, of the size and nature of the apparatus and of the mechanical operations involved". (Felt, 1958)

The research was carried out over a period of many years starting in 1954. The findings led to the updating of the Bureau of Reclamation's test procedure for Test Designation E-12 to include a method involving a vibratory table. This was included as Part B of Test E12 in the first edition of the Earth Manual in 1966 and formed the basis of what was to become ASTM Designation D:2049-69, the relative density method. The research findings were summarized in Earth Laboratory Report No. EM557 (Pettibone, 1961) and EM 697 (Hardin, 1965). Results of preliminary investigations were published by the chairman of the sub-committee, E.J. Felt (1958), and an interim paper was presented by Pettibone and Hardin as part of ASTM STP 377.

2.5.2 Laboratory methods of compacting granular materials, Felt (1958).

This paper by Felt is an important milestone because it indicates the large scale of the investigation and some of the major conclusions drawn from the tests.

The subsequent US preference for the use of a table vibratory rather than the vibrating tamper has its origin here. The research was linked to the concept of relative density which was applicable to cohesionless soils. The aim was, "... to develop methods for determining the maximum density of granular soils". Such maximum density was to be, "..... the absolute maximum possible with any particular soil".

Six granular materials were compacted by each of eight contributing research institutions according to the method of their choice. A total of twelve different test methods were used by the investigators.

The methods used varied widely, employing both vibration and free fall methods for imparting compactive effort. Mould sizes varied from a truncated metal cone to cylinders with diameters ranging for 50 - 280 mm. Free fall methods typically employed a height of 457 mm while vibratory methods employed frequencies of 3 500 - 14 000 rpm at varying amplitudes. Compaction was done under surcharge pressure applied by a dead mass or spring loaded plates with pressures varying from 0 - 290 kPa. Generally the methods used differed greatly and could be compared only as one method against the other.

The six materials tested included a fine sand, medium sand, coarse sand, well-graded sand-gravel, well-graded crushed rock and a uniformly-graded crushed rock.

All materials were classified as cohesionless . For the three sands the percentages passing the 0.075mm sieve were, in all cases, less than 7%. The sand-gravel and crushed rock materials contained from 8 to 22% particles passing the 0.075mm sieve. Both the sands and the gravels were "free-draining" and the fines were non-plastic in all cases.

Moisture-density tests according to the standard AASHTO method ASTM Designation: D698-42 indicated optimum moisture contents of 7 to 15% for the sands and 5 to 8% for the coarse grained materials.

The following conclusions were drawn by Felt from the co-operative investigators.

i) The cohesionless sand soils did not compact satisfactorily using ASTM Designation: D698-42 but readily compacted to uniformly-high densities with vibratory-table methods.

ii) Reasonably high densities were obtained with a vibrating tamper and with a compacting rammer having the same diameter as the mould.

iii) Vibrating free-drop and free fall methods did not produce satisfactory results.

iv) The three sand soils were compacted to maximum dry density by vibration when they were either dry or saturated. At moisture contents between these limits, lower densities were obtained.

v) In the compaction of the coarse-grain soils vibratory-table methods were more effective where water was present, than in the dry. The best results were obtained when the mould permitted drainage from the bottom and the sample was placed in the mould at a moisture content somewhat above saturation. Drainage during the test reduced the water content to a value commensurate with the voids at maximum density.

vi) Further research into the methods employing a vibratory table were recommended.

The author proposed a test method upon which further research could be carried out. The test was aimed at achieving acceptable maximum densities with both fine-grain and coarse-grain granular materials. To this end two different size moulds were suggested, one 152mm in diameter and 152mm high (CBR mould) for material with particle sizes up to 25mm and another 279mm in diameter and 231mm high for materials with particle sizes up to 64mm.

The time for vibration required was estimated as "reasonable to obtain maximum density .... probably less than about 20 minutes".

The bottom of the mould was to be designed to permit drainage without the significant loss of fines.

Materials were to be tested under both dry and saturated conditions in order to establish which gave the highest result.

2.5.3 Earth Laboratory Report EM 557 (Pettibone, 1961)

The US Bureau of Reclamation was one of the co-operators of Sub-committee R-3 of Committee D-18 of the ASTM. Report No. EM557 sets out the Bureau's test programme as a co-operator as well as testing on some additional materials.

Tests were performed with 3 sizes of table-type vibrators, 2 sizes of foundry type vibrators (these clip onto the side of the mould), 2 sizes of immersion-type vibrators and 1 pneumatic jolting device. (Note: no vibratory rammer or tamper was tested despite the "reasonable" results reported by Felt).

Surcharge pressure was applied during testing by means of a spring, a handheld deadweight surcharge or a guided deadweight surcharge.

The two materials, a fine sand and a coarse sand, were tested either in the oven dry or saturated condition.

Three sizes of cylindrical mould were used depending on the maximum particle size of the soil. These included the 152mm diameter and 279mm diameter moulds recommended by Felt (1978) which had capacities of 0.1 (0.0028 m^3) and 0.5 ft^3 (0.0142 m^3) respectively, as well as a 1.0 ft^3 (0.0283 m^3) measure.

Tests were grouped as four series. The following results were drawn from the first series:

i) Better reproducibility of maximum density results was obtained by completely filling the measure prior to the start of vibration rather than simultaneously vibrating and filling the measure.

ii) An electro-magnetic table vibrator provided better compaction than both the immersion-type or the foundry type vibrators which were attached alternately to each side of the mould.

iii) For the two materials tested, higher densities were obtained using oven dry than saturated material.

iv) Higher densities were achieved when material was vibrated with a deadweight surcharge than without.

On the basis of the results from the first tests, a second series of tests was performed on the six soils forming part of the co-operative investigation. A table vibrator operating at 3600 rpm with an average amplitude of 0.305mm was used. The mould was fastened firmly to the table by means of a yoke after positioning the surcharge weight. The sample was vibrated for 8 minutes. An alternate method was to vibrate the soil 4 minutes without surcharge, add the surcharge, and vibrate an additional 4 minutes.

The surcharge pressure applied in the form of loose metal plates, varied from 2.0 to 11.5 kPa.

The following conclusions were drawn:

i) The 152mm diameter mould (CBR-mould) was suitable for compaction of material with up to 19mm particles, while the 279mm diameter mould (0.5 ft^3) could be used for materials with maximum particle size up to 37.5mm.

ii) The highest maximum densities were achieved under surcharge pressures of $\pm$ 7 kPa.

iii) Higher maximum densities were obtained using dry material rather than saturated material for all six soils tested.

iv) A practical length of vibration time was 8 minutes.

v) Vibration of the sample under no surcharge for any length of time appeared to have no compacting effect.

vi) The crushed stone material with 15% passing the 0.075mm sieve yielded a significantly lower density when compacted under saturated than under dry conditions. The material showed slight segregation during testing. Better density could be achieved by the Standard AASHO test for crushed stone materials than under vibration. It was suggested that the crushed stone materials gave unreliable results when compacted under vibrating conditions.

In the third series of tests, various combinations of vibrator type and method of applying surcharge pressure were investigated.

A deadweight surcharge system and a spring surcharge system were used. The deadweight surcharge was composed of a series of plates. The amount of deadweight surcharge varied from 0 to 55 kPa. In the spring surcharge system the pressure was maintained on a spring by means of a hydraulic jack. The surcharge pressures applied by this method varied from 0 to 250 kPa.

One table vibrator used vibrated at 3600 rpm with an average amplitude of 0.355mm. The power was varied for some of the tests, but most of the tests were run with the maximum power available. (Varying the power input would alter either the frequency or amplitude of vibration or both depending on the characteristics of the machine). A second table vibrator with a frequency of 3 600 rpm and an average amplitude of 0.305mm was operated at 85% of maximum power. A pneumatic table jolter operating at 100 jolts per minute and an amplitude of 37.5mm and two foundry type vibrators attached to the side of the mould were also used.

The time of vibration was 8 minutes.

The following conclusions were drawn from the third series of tests:-

i) The foundry type vibrators attached to the side of the mould produced significantly lower compaction than the vibratory tables.

ii) The pneumatic jolter produced densities slightly less than the electro-magnetic table vibrators. The particular device used required a compressed air supply at 620 kPa, which was very noisy and limited the surcharge to 55 kPa.

iii) The deadweight surcharge produced greater maximum densities than the spring-applied surcharge.

iv) Both table vibrators operated at 3600 rpm, but the one was capable of maintaining a higher amplitude. The larger machine operating at full power produced the best results.

A fourth and final series of tests was carried out using the larger of the two table vibrators. Three sizes of mould of 0.1; 0.5 and 1.0 ft^3 volume were used with deadweight surcharges ranging from 7 to 55 kPa.

For those tests performed with oven dry soil, the mould was filled in two lifts. Each lift was vibrated 8 seconds without surcharge, after which the surcharge was applied and the loaded specimen vibrated for a further 8 minutes.

In tests carried out with saturated soil, the specimen was vibrated during filling and the surcharge applied before vibrating under load for 8 minutes.

The following conclusions were drawn:-

i) The optimum surcharge varied between 7 and 48 kPa for the 152mm diameter mould and between 7 and 21 kPa for both the larger moulds.

ii) Under the test conditions higher densities were achieved in some soils with the smaller mould.

iii) An increase in the surcharge above the optimum amount caused a decrease in density.

iv) The use of saturated rather than oven dry material produced a significant increase in density for soils with 19mm or larger maximum size particles.

v) Greater maximum densities were obtained where specimens were not vibrated prior to surcharging.

Based on all the tests carried out under this programme the following general conclusion were drawn:

i) The greatest maximum densities were achieved with an electromagnetic table-type vibrator with a 14 kPa deadweight surcharge, vibrated for 8 minutes.

ii) A 152mm diameter (CBR) mould should be used for soils with particle sizes up to 37.5mm. A 279mm diameter mould should be used with soils with up to 75mm maximum particle size.

iii) The maximum density for sand was achieved with oven dry material, while the maximum density for gravelly soil was achieved with saturated material.

2.5.4 Interim report by Pettibone and Hardin, 1965.

Pettibone and Hardin were involved with the US Bureau of Reclamation's work described above. The research findings in Report No. EM 557 had shown conclusively that higher densities could be obtained for cohesionless soils using vibratory methods, than with impact methods but that with no single standard method could be developed, since no single method gave a maximum density for all soils. The authors carried out further tests in order to gain a better understanding of the effects of magnitude of surcharge, time of vibration, amplitude of vibration and water content.

Two soils were chosen for these tests; a poorly graded fine sand and a poorly graded sand-gravel mix with a 75mm maximum particle size.

Tests were carried out on two table-type vibrators both having frequencies of 3600 rpm but differing in amplitude characteristics. For both tables the amplitude could be adjusted by means of a rheostat. A 14 kPa deadweight surcharge was applied. Most tests were performed with oven dry material, but some were run with initially saturated soil. The effect of placing soil in the mould at various initial densities was investigated.

Measurements taken during the tests indicated that the amplitude was reduced with increasing load on the table. The decrease with increasing load was most rapid for loads in excess of 90 kg. It was also apparent that amplitude was related to the size, shape and arrangement of the load. There existed therefore an interaction between the load and the amplitude.

The following conclusions were drawn:-

i) The effect of the density before vibration on the maximum density appeared not to be significant.

ii) There was no significant difference between the densities achieved with oven dry or initially saturated material.

iii) The amplitude of vibration appeared to be the most significant variable affecting soil density. The highest maximum densities were obtained with the higher amplitudes. A further increase in amplitude was likely to improve maximum densities.

iv) When the surcharge was applied to the soil by a spring-loaded system the load could be increased without altering the total load on the table. The maximum densities achieved in the range, 3.5 to 248 kPa, were independent of the surcharge in the spring-loaded system.

v) The apparent effect of surcharge and mould size on the maximum density with the deadweight-system was probably due to the load-amplitude characteristics of the vibrator used.

vi) Solid surcharge weights could give a radically different relationship of load and amplitude than loose plates.

2.5.5 Earth Laboratory Report EM-697 (Hardin, 1965)

Hardin continued the Bureau of Reclamation's research into the mechanisms of laboratory vibratory compaction.

Variables considered in the investigation were time of vibration, density before vibration, amplitude, frequency and whether material should be oven dried or initially saturated.

A certain number of the tests were carried out with electromagnetic table-type vibrators similar to those used in the investigation reported on in Report No. EM-557. The frequency of both the vibrators was fixed at 3600 rpm but the amplitude could be varied by means of a rheostat. Other tests were conducted with an eccentric weight-type vibrator. The amplitude and frequency of this machine could be varied by a rheostat. The amplitude and frequency could however not be varied independently of each other.

A single-piece deadweight surcharge of 14 kPa was applied in all tests.

All three soils tested were poorly graded and consisted of two fine sands with more than 10% smaller than 0.075mm, and a sand-gravel with a maximum particle size of 76mm and no fines.

A 152mm diameter (CBR) mould was used for the sands and a 279mm diameter mould for the sand and gravel.

The tests with oven dried material were run for 16 minutes with density determinations every 4 minutes.

Tests on initially saturated specimens were run for a total of 20 minutes with density determinations every 4 minutes. The tests on initially saturated material were run after the optimum rheostat setting (i.e frequency and amplitude) had been determined with oven dried material.

For the electromagnetic vibrators the amplitude generally decreased as the load on the vibratory table was increased, but for the eccentric weight vibrator amplitude was independent of the load. The amplitude-load response of the machine was significantly different for a single piece surcharge as opposed to a loose-plate system.

The following tentative conclusions were drawn on what the author felt was insufficient data to permit definite conclusions:-

i) The density obtained by vibration was independent of the frequency in the range 3600 to 8400 rpm at an amplitude of 0.13mm $\pm$, but dropped rapidly for frequencies below 3600 rpm.

ii) The density obtained by vibration was independent of the amplitude in the range 0.25 to 0.51mm at a frequency of 3600 rpm, but dropped rapidly for amplitudes less than 0.25mm.

The following definite conclusions were drawn:-

i) The increase in density for times of vibration greater than about 6 minutes was "insignificant" for the soils and equipment used.

ii) The initial density did not have a significant effect on the final density obtained by vibration.

iii) The frequency and amplitude of vibration did have an effect on the density obtained. Results suggested certain limiting ranges.

iv) Degradation of a weakly cemented dune sand during vibration was found to increase the maximum density.

The laboratory vibratory test included in ASTM Designation: D2049-69 for determining the maximum dry density of cohesionless material was based on the work by the Bureau of Reclamation from 1954 to 1965. After publication in 1969 the test was used in practice for some years. In 1972 a special symposium was held at the 75th Annual Meeting of the ASTM (cf Section 2.2.2) where the relative density concept and its uses was evaluated. The advances made since the USBR's research program were presented here. Three papers of direct relevance to cohesionless soil presented at this symposium are summarized in 2.5.7 to 2.5.9.

In the interim, Hoover et al (1970) published a paper entitled : "Degradation control of crushed stone base course mixes during laboratory compaction". The research summarized in 2.5.6 is notable because the research appears to have been carried out without reference to the USBR's work and because an attempt was made to not only compact to Standard AASHO density, but also to Modified AASHO density.

2.5.6 Hoover, Kumar and Best (1970).

In order to assess the degradation of crushed stone base courses during laboratory compaction the authors carried out a number of types of tests including Standard and Modified AASHO tests and a test with a table vibrator.

The authors make no reference to the Bureau of Reclamations' research into a maximum density test using a table vibrator for cohesionless soils and appear to have taken an independent view.

Three crushed stone basecourse materials were tested. The maximum particle size was 19 mm in all cases. The materials included:-

i) No. 1 A weathered, moderately hard quarried limestone with 8.4% less than 0.075mm size and a plasticity index of 2.

ii) No. 2 A hard quarried limestone with 10.2% less than 0.075mm size without plasticity.

iii) No. 3 A hard dolomite with 5.9% less than 0.075mm size without plasticity.

The authors make no reference to the drainage characteristics of the soils but judging from the percentage of fines in soils 1 and 2 and comparing these with the curves for the influence of particle size distribution on permeability by Cedegren (1972) in Figure 2.18 these soils are definitely not "free-draining".

Most of the tests were carried out on soil No 1. The soil was placed in a 152mm diameter mould at the optimum moisture content determined with Standard AASHO compaction. The electromagnetic table vibrator was run at a constant 3600 rpm but the amplitude was adjusted during testing.

Tests were carried out with vibration periods of ½, 1 and 2 minutes under single-piece deadweight surcharges of 3.7, 6.1 and 8.6 kPa. Measurements showed that for a given amplitude control dial setting the measured amplitude of vibration decreased with an increase in total load on the table. A maximum amplitude of 0.915mm was measured under the 3.7 kPa surcharge, while a minimum of 0.320mm was measured under 8.6 kPa.

Maximum densities achieved were of the order of Standard AASHO maximum dry density for the 8.6 kPa surcharge, but more erratic and generally lower for the lighter surcharges.

It was concluded that the optimum combination of the variables comprised, the highest (i.e. 8.6 kPa) surcharge with the maximum amplitude under this load and a vibration time of 2 minutes.

Tests similar to those conducted with soil No. 1 were carried out with soils No. 2 and No. 3. With the previously mentioned optimum combination of variables controllable maximum densities consistently 2 - 3% above the Standard AASHO maximum dry density were achieved.

In order to achieve Modified AASHO maximum dry density the surcharge weight was increased to 58 kPa while the other variables were kept as before. The Modified AASHO maximum dry density was achieved under these conditions. It was found however that to achieve this density with soil No. 1 the moisture content of the specimen prior to vibration had to be 1,1% greater than for the Modified AASHO test (i.e. the OMC was not the same for the two methods). For soils No. 2 and No. 3 marginally greater percentages of water were required to achieve maximum density under vibration than with the Modified AASHO test.

2.5.7 Laboratory studies of maximum dry densities of cohesionless soils by Johnston, (1972).

The author carried out a series of compaction tests on a table vibrator on a sandy gravel, several gravelly sands, a silty-sandy gravel and poorly-graded sand. The tests were carried out according to the method recommended in the Earth Laboratory Report No. 557 (cf Section 2.5.3). All tests were carried out on oven-dry materials.

The author concludes that the maximum dry density of a cohesionless soil is a function of its grain-size distribution and its specific gravity. The grain-size distribution and the maximum density are correlated for subangular to rounded granular soils on the basis of the tests. A feature of the soils used for the correlation is that less than 5% of material is retained on the 0.075mm sieve in all cases.

The coefficient of uniformity, C_u, is used to indicate grain-size distribution. (cf Section 2.3.3.1)

The coefficient of uniformity for each soil was plotted on a logarithmic scale versus the maximum dry density on an arithmetic scale. (cf Figure 2.5). The plot is for a constant specific gravity of 2.65, though dry densities for any other value of specific gravity can be found by multiplying by the ratio of the desired specific gravity to 2.65.

A second conclusion which the author comes to on the basis of his tests is that the amplitude of the table vibrator exerts considerable influence on the final dry density. He suggests that there is an optimum amplitude of vibration for each type of granular material. He adds however that this optimum amplitude is probably also a function of the type of table and the surcharge. The plot in Figure 2.19 shows the variation of amplitude with maximum dry density for two sands tested under the particular surcharge on the available table vibrator. The plots indicate an optimum amplitude in each case of $\pm$ 0.01 inches (0.25mm).

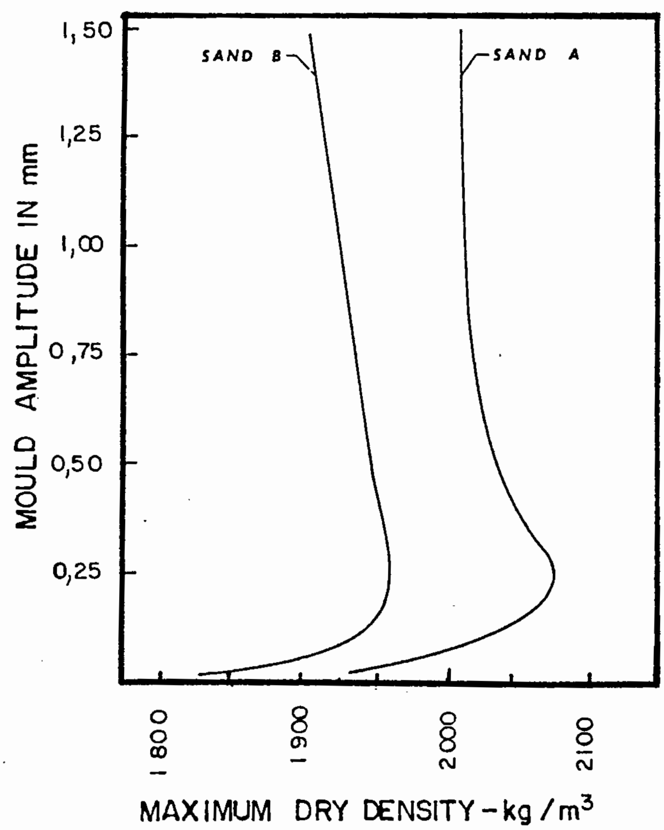


FIGURE 2.19:- RELATIONSHIP OF MOULD AMPLITUDE VERSUS MAXIMUM DRY DENSITY. (AFTER JOHNSTON, 1972)

2.5.8 Maximum density determination of subbase materials by Cumberledge and Cominsky, (1972).

The authors carried out a series of tests on three subbase materials - gravel, limestone and slag. The aim of the tests was to assess the interaction between aggregate type, mould size, surcharge pressure and duration of vibration.

The tests were conducted according to ASTM D 2049-69 except that the amplitude and surcharge pressures were varied. A statistical analysis was carried out to determine which of the effects of the parameters considered had a statistically significant impact on the maximum dry density obtained.

The following points of interest emerged from the tests:-

(i) The interaction effects of subbase type, mould size, amplitude of vibration, surcharge pressure and duration of vibration were all found to be highly significant on maximum dry density.

(ii) There was no significant difference in maximum dry density between specimens vibrated in the air dried state compared to initially saturated conditions. At intermediate moisture contents the resultant densities were significantly lower.

(iii) The type of subbase material had an effect even if similar gradings were used. The variation in densities between one material and another under similar test conditions could be attributed partly to the variation in particle shape. Moreover for material with a high percentage of voids in the material aggregate, significantly lower densities could be expected.

(iv) The interaction between mould size and subbase type was not found to be significant. Maximum density was however affected when different mould sizes were utilized for the same material. Cylindrical moulds were found suitable, while a rectangular mould resulted in unreliable and low densities.

(v) By far the most significant interaction effect was between amplitude and surcharge pressure. When an interaction is statistically significant, the corresponding main effects (in this case amplitude, surcharge pressure and specimen type) cease to have importance on their own. The surcharge in the tests was of the deadweight type. The effect of amplitude on the resultant density was markedly dependent on the level of applied surcharge. When quoting the effect of amplitude, it was suggested therefore that it was necessary to indicate the magnitude of applied surcharge pressure also.

For a given table vibrator and material there existed an optimum combination of amplitude and surcharge pressure to produce maximum dry density. In the tests carried out, density increased with surcharges up to 14 kPa. At a pressure of 21 kPa the densities decreased, either because increased effective stress made particle movement more difficult or because the larger deadweight affected the amplitude of the vibratory table.

(vi) There was significant interaction between duration of vibration and surcharge pressure. Generally for higher surcharge pressures longer periods of vibration were required before the maximum density stabilized.

2.5.9 Compaction of sand on a vibrating table without surcharge by Dobry and Whitman, (1972).

The authors made a study of the compaction behaviour of a dry sand on a vibrating table without applying any surcharge pressure. The soil was a quartz sand with subangular grains, particle sizes between 0.25 and 2mm and a uniformity coefficient (C_u) of 1.7. The specific gravity of the individual particles was 2.64. The minimum density was 1388 kg/m³ while the maximum density estimated from various tests was 1640 kg/m³.

The table vibrator produced vertical oscillations of approximately sinusoidal shape with a double amplitude up to 3.8mm within a range of frequencies of 10 to 60 Hz. The amplitude was set for any one test but the frequency could be changed during operation. Accelerations of the vibrating table were measured by means of an accelerometer.

Three cylindrical moulds of comparable dimensions but composed of different materials were used. The following parameters were studied.

i) peak accelerations (a_{max}) of from 0 to 3 g.
($a_{max} = (2\pi f)^2 A$ where A = amplitude and f = frequency)

ii) double amplitudes (2A) of 0.635, 1.270 and 3.810 mm

iii) three mould materials

iv) sample heights of 75, 150 and 250 mm

The following conclusions were drawn on the basis of the test results:

i) Mould material type was unimportant provided the peak acceleration $a_{max} > 1$ g (cf Figure 2.22).

ii) There existed a distinctive relationship between maximum density and peak acceleration. Referring to Fig 2.20 where dry density is plotted against peak acceleration in g's the following is noticeable:

- Below 0.9 g there is little densification
- Most of the densification is produced in the range 0.9 to 1.1 g
- In all cases there is a well defined peak density and a corresponding optimum peak acceleration which ranges from 1.1 to 1.3 g.
- The loosening of the sand if vibrated above the optimum acceleration after maximum density has been achieved is not large.
- At some point between 1.3 and 2 g the loosening process stops and either the density stabilizes or increases again.

iii) The amount of densification produced below 1 g varied widely from one series of tests to the next. Densities measured for peak accelerations of 0.93 g ranged from 20 to 70 % relative density. Conversely for peak accelerations of 1.1 g relative densities were 83 $\pm$ 3%. The density achieved above 1 g was therefore notably constant, being independent of mould type and specimen height as well as amplitude of vibration.

iv) Above accelerations of 1 g the sand jumps free of the mould and subsequently falls back causing an impact of the soil specimen.

The authors' interpretation of the densification process is summarized in Fig 2.21, which is a plot of peak acceleration in g's against frequency. There are two processes leading to densification. Firstly when a_{max} = 1 g there is rapid densification to an equilibrium density of about 80% relative density which is independent of initial density, frequency and sample height. Densification occurs because the initial intergranular stresses are released allowing particles to rearrange in a denser packing. The density achieved at this stage is the maximum possible by simply releasing potential energy.

Secondly, above a_{max} = 1.1 g, further densification occurs as a result of impacts, which provide the force necessary to overcome frictional resistance from surrounding grains. Impact velocity and dry density were shown to be related by Selig (cf Figure 2.23). The authors show from their experiments that for a given frequency, impact compaction causes densification up to given peak acceleration above which the soil is loosened by impacts. The conditions giving peak density on the graph are indicated by the intersection of the line of constant amplitude with the line of spalling i.e. for a given frequency there exists an optimum amplitude to yield maximum density.

v) The authors suggest that saturating the sand and adding surcharge weight could improve the density by reducing spalling.

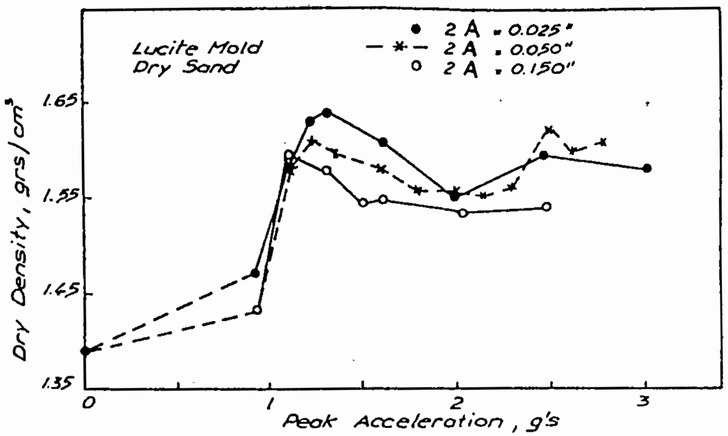


FIGURE 2.20:- TYPICAL RESULTS FOR DENSITY AS FUNCTION OF ACCELERATION

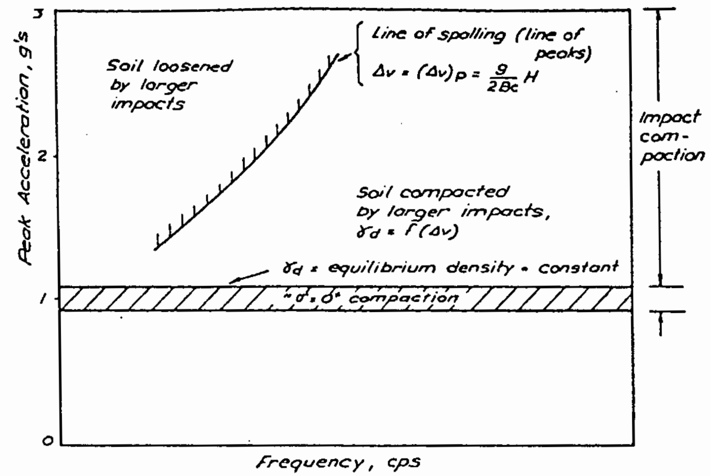


FIGURE 2.21:- GENERAL COMPACTION BEHAVIOUR (AFTER DOBRY AND WHITMAN, 1972)

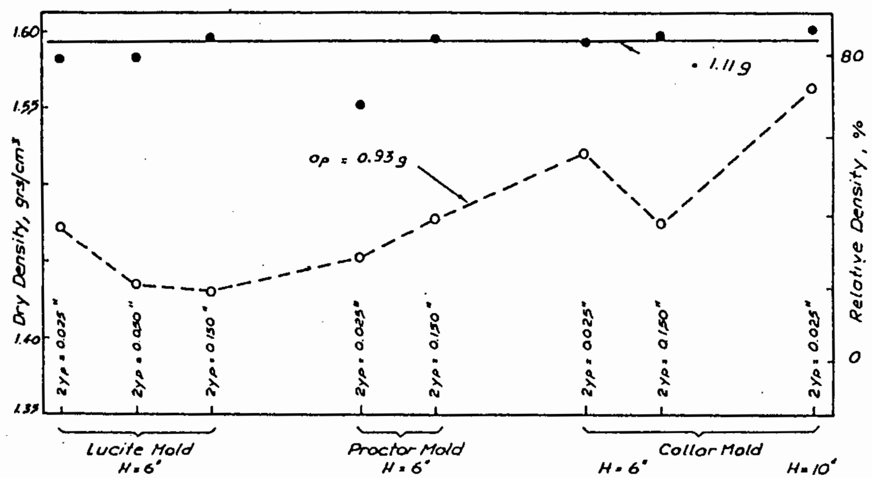


FIGURE 2.22:- EQUILIBRIUM DENSITY FOR a_{max} = 1.1 g. (AFTER DOBRY AND WHITMAN, 1972)

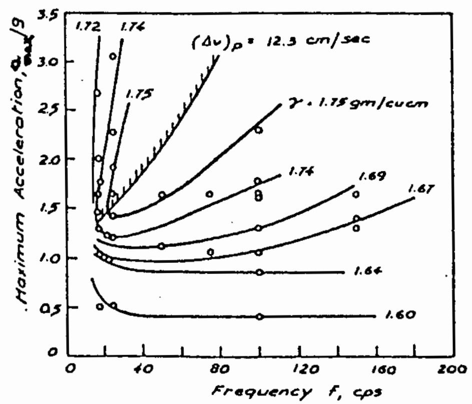


FIGURE 2.23:- DENSIFICATION RESULTS: DRY SAND (AFTER SELIG, 1963)

2.5.10 Research by S Pisarczyk carried out at the Technical University of Warsaw, (1980).

The author carried out research on "the laboratory testing of coarse-grained soil compactibility with application of vibration" It is evident that the author was largely unaware of research in this field in the West, and as a result the work represents a fresh and independent approach.

The soil used was a coarse-grained sand with angular to subrounded grains with less than 5% passing the 0.075mm sieve.

A diagrammatic illustration of the equipment used is shown in Figure 2.24. The table could be vibrated at frequencies of 22, 34 and 74 Hz. The amplitude could be adjusted by dial gauge in the range 0.28 to 0.98mmm. The surcharge pressure maintained by the spring could be up to 150 kPa. The specimen was vibrated for 1 minute under no surcharge before the spring surcharge was applied.

It was found that the time of vibration, thickness of the vibrated layer, surcharge pressure, amplitude and frequency of vibration and the moisture content of the soil influenced the compaction.

The following conclusions were drawn:-

i) For the particular material an optimum density was achieved at a frequency of 74 Hz and an amplitude of 0.4mm under a spring surcharge load of 150 kPa. This density was equivalent to Modified AASHTO maximum dry density.

ii) The maximum dry density was achieved at a moisture content close to saturation. This density was slightly higher than that achieved with vibration in the dry state.

iii) The time of vibration to achieve a stable condition was approximately 12 minutes.

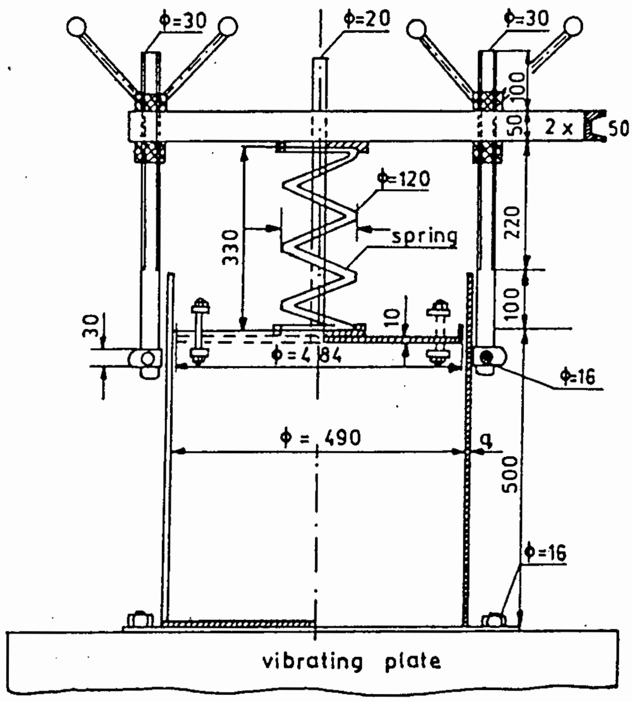


FIGURE 2.24 :- APPARTUS FOR VIBRATORY SOIL COMPACTION (PISARCZYK, 1980)

iv) The diameter of the mould should be at least 5 times the maximum specimen particle size.

2.6 The mechanism of laboratory vibratory compaction of material exhibiting some cohesion.

Krizek and Fernandez, (1971) and Townsend, (1972) have carried out vibratory tests on soil exhibiting cohesion and "non-free draining" characteristics. The former carried out tests on sand-clay mixtures which therefore constituted cohesive material which is certainly not "free-draining". The latter carried out tests on sands with varying amounts of plastic fines. This material may therefore only have exhibited apparent cohesion.

2.6.1 Vibratory density tests with varying amounts of plastic fines by Townsend, (1972).

The author undertook a comparison of the maximum vibrated density achieved with ASTM: D2049-69 the maximum density achieved with Standard AASHTO compaction on sands with varying amounts of fines.

The aim of the investigation was to assess the effects of gradation, percentage and plasticity of fines and moisture content on the compaction of granular soils.

Two natural sands were used in the study. The first was a subangular to subrounded concrete mortar sand and the second a subangular to angular sand. Both sands were sieved to remove all natural fines. A soil type with all particles passing the 0.075mm sieve size was added to each of the sands to make up predetermined gradings. Samples of each of the sands with 0, 9.1, 16.7 and 23.1% fines respectively were made up.

The soils were compacted according to the Standard Corps of Engineers' impact test which is roughly comparable to ASTM: D698-70 (cf Table 2.1). Oven dry material was vibrated in a CBR mould at a frequency of 50 Hz (sic.) and an amplitude of 0.48mm under a deadweight surcharge of 14 kPa. The duration of vibration was 8 minutes. Tests were also carried out on the saturated and moist samples.

The following conclusions were drawn with respect to the vibrating method:-

i) The maximum densities of sands were sensitive to the gradation and percentage of fines present. More fines could be accommodated in a uniform sand and a higher density achieved than in a well-graded sand.

ii) Moisture and plasticity of fines were interrelated factors which greatly influenced the compaction characteristics. For low plasticity fines saturation facilitated densification. Conversely for more plastic fines, moisture caused adhesion of fines to the sand grains restricting densification.

iii) Compaction of well-graded sand with fines was more affected by moisture than a uniform sand with fines.

iv) When material was vibrated in the oven dry condition where the effect of plasticity was of no consequence, the misleading conclusion could be drawn that sands with up to 20% fines could be compacted satisfactorily with their particular vibratory testing technique.

v) There was a correlation between grading, percentage of fines and maximum density for vibratory compaction.

2.6.2 Vibratory densification of damp clayey sands by Krizek and Fernandez, (1971).

These tests were conducted to study the effect of water content and varying amounts of cohesive fines on vibratory densification. Air-dry and moist specimens of sand and sand-clay mixtures were tested at various amplitudes and frequencies under three conditions of confining stress. Terminal density was defined as that density achieved after 5 minutes of vibration.

Two natural soils were used. These were tested individually and as combinations composed of various percentages of each. The first natural soil known as Ottawa sand was uniformly-graded and rounded with a mean grain size of 0.38mm, a uniformity coefficient of 1.6 (C_u) and a maximum dry density (Modified AASHTO) of 1725 kg/m³ at an optimum moisture content of 11.0%. The second natural material known as Grundite was a silty clay with a trace of very fine sand. It had a liquid limit of 48, a plasticity index of 24 and a maximum dry density (Modified AASHTO) of 1623 kg/m³ at an OMC of 18.5%. The clay fraction of this material, which consisted primarily of illite represented about 60% of the material and had a LL of 93 and PI of 65. The following mixtures were made with the two natural materials.

i) MIX 10 (90% sand and 10% Grundite) with Modified AASHTO maximum dry density = 1830 kg/m³ at an OMC = 8%

ii) MIX 20 (80% sand and 20% Grundite) with a Modified AASHTO maximum dry density of 1930 kg/m³ at an OMC of 9%

iii) MIX 30 (70% sand and 30% Grundite) with a Modified AASHTO maximum dry density of 1984 kg/m³ at an OMC of 9.5%

A schematic representation of the equipment used is shown in Figure 2.25. The cylindrical mould was 475mm high and 305mm in diameter. Surcharge pressure was maintained by means of a bellows-operated piston connected to a large tank of compressed air. The peak-to-peak amplitude of vibration could be varied between 0 and 3.81mm and the frequency range extended from 10 to 38 Hz. Surcharge pressures of 0.23 and 45 kPa were utilized.

The series of tests which were conducted on air dried samples of the "pure" sand and Grundite soils were vibrated at double amplitudes of 0.635, 1.27 and 2.54 mm and at frequencies of 10, 15, 20, 30 and 35 Hz. Sand-dry mixtures were vibrated only at such frequencies and amplitudes as produced accelerations in excess of 1 g (cf Dobry and Whitman, 1972).

The test series conducted with sand and sand-clay mixtures were tested at an average moisture content of 4.5% over a range of accelerations and over a range of moisture contents with a double amplitude of 2.81mm and a frequency of 20 Hz. The tests with variable water content were carried out under zero surcharge pressure.

The experimental results are represented graphically in Figs. 2.26 to 2.30. Based on these test results the following was concluded:-

i) Significant vibratory densification did not occur at peak accelerations less than 1 g.

ii) Most of the vibratory densification took place within approximately 1 minute. A slightly longer time was required for densification to stabilize for soils with higher clay contents.

iii) No direct dependence was found between terminal vibratory density and the amplitude or frequency of vibration.

iv) For air dried soils under zero surcharge pressure, accelerations greater than about 2 g caused a slight decrease in density. For soils which were confined this principle was not found to apply. The density of the damp soils tended to increase with an increase in acceleration for all surcharge pressures.

v) Except for the Grundite, the application of a surcharge pressure reduced the terminal vibratory density for the air dried and damp sand and for the air-dry soil mixtures. This trend was not observed for the damp soil mixtures for accelerations in the range of 1 g to 3 g. In the case of air-dry Grundite no significant densification was observed under vibration even at high accelerations.

vi) An increase in the percentage of cohesive fines in an air-dry soil reduced the maximum density that could be obtained by vibration, the effect being accentuated when the soil was subjected to a surcharge pressure.

vii) Vibratory densities of the order of 95% and 85% of Modified AASHTO maximum dry density could be obtained for sandy soils with up to 30% cohesive fines under surcharge and under unconfined conditions respectively, provided the soil was air-dry.

viii) Water content had the greatest influence on the vibratory densification process. Relatively small differences in moisture content could lead to large differences in density.

ix) For accelerations in the range of 1 g to 2 g the damp sand exhibited a percent compaction between 85% and 90% of Modified AASHTO maximum dry density, while the damp sand-soil mixtures which contained from 10% to 30% cohesive fines yielded a percentage of compaction of the order of 65% to 75%.

2.7 A mathematical model for vibratory roller behaviour in the field by Yoo and Selig, 1977.

The authors examined the mechanisms by which vibratory smooth-drum rollers achieve compaction in the field and the factors which influence the results. The conclusions were based on a mathematical model representing the response of the soil-machine system, laboratory model roller tests and full-scale field tests. The research showed that the amount of compaction with a vibratory roller could be subdivided into two components, one related to the static ground contact force per unit width of roller, and the other to the amplitude of drum vibratory displacement and the ratio of vibration frequency to travel speed. The dynamic mechanism causing compaction was described as "the accumulation of residual strain produced by cyclic soil straining as a result of drum oscillation".

Two important clarifying statements are made in the introductory remarks:

> "One of the reasons for the inadequate state of understanding is that field and laboratory research in the past have tended to focus on either the machine or the soil, but not both, in spite of the fact that it is the combined characteristics of the machine and the soil which determine the amount of compaction".

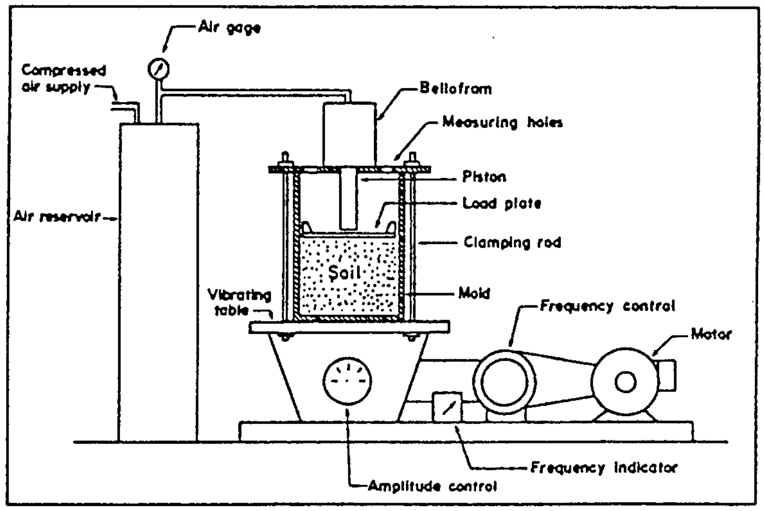


FIGURE 2.25:- SCHEMATIC DIAGRAM OF TESTING APPARATUS (KRIZEK AND FERNANDEZ, 1971)

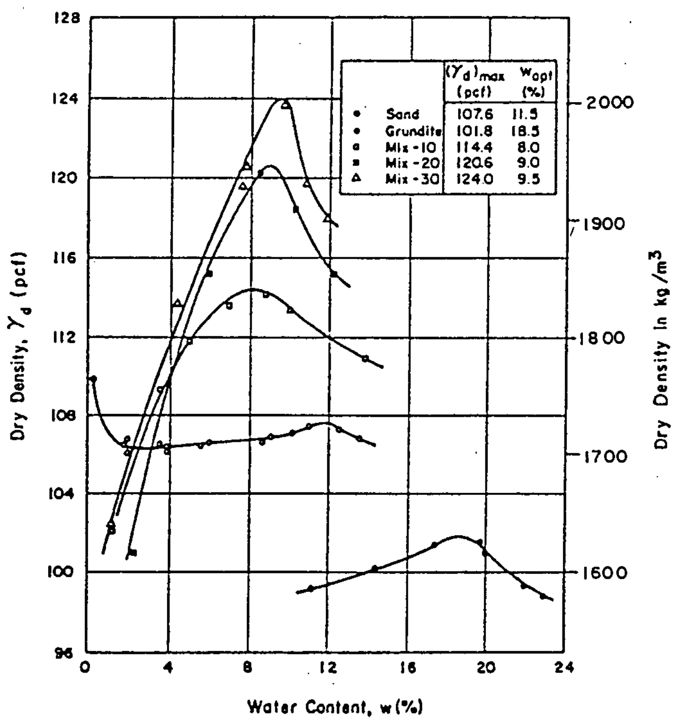


FIGURE 2.26:- MOISTURE-DENSITY RELATIONSHIPS OBTAINED IN ACCORDANCE WITH ASTM D 1557-66T - MODIFIED PROCTOR TEST (AFTER KRIZEK AND FERNANDEZ, 1971)

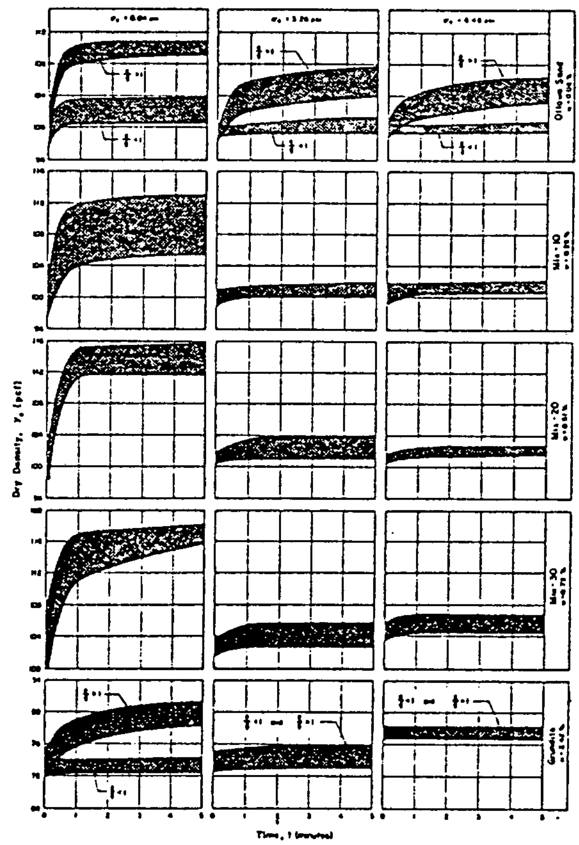


FIGURE 2.27:- TIME RATE OF VIBRATORY DENSIFICATION FOR AIR-DRY SOILS (KRIZEK AND FERNANDEZ,1971)

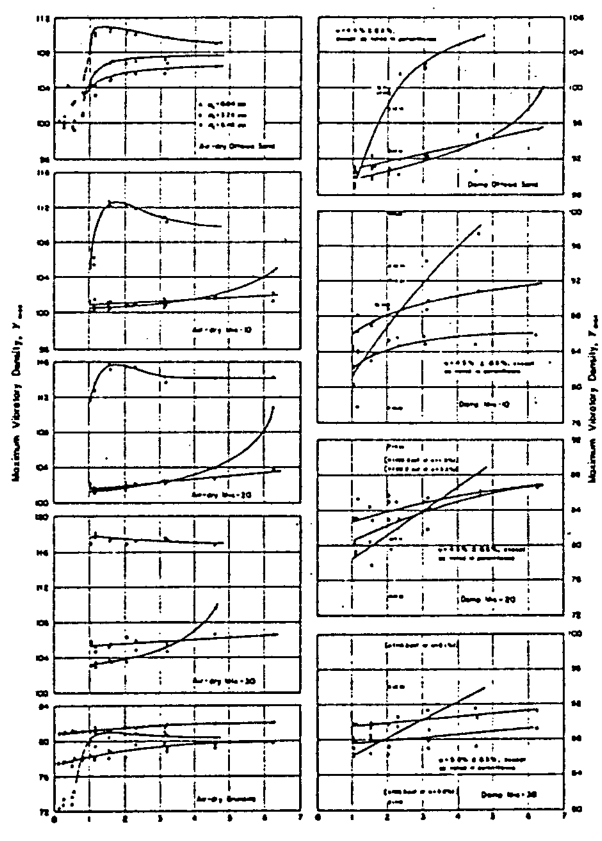


FIGURE 2.28:-MAXIMUM VIBRATORY DENSITY VERSUS ACCELERATION RATIO (KRIZEK AND FERNANDEZ,1971)

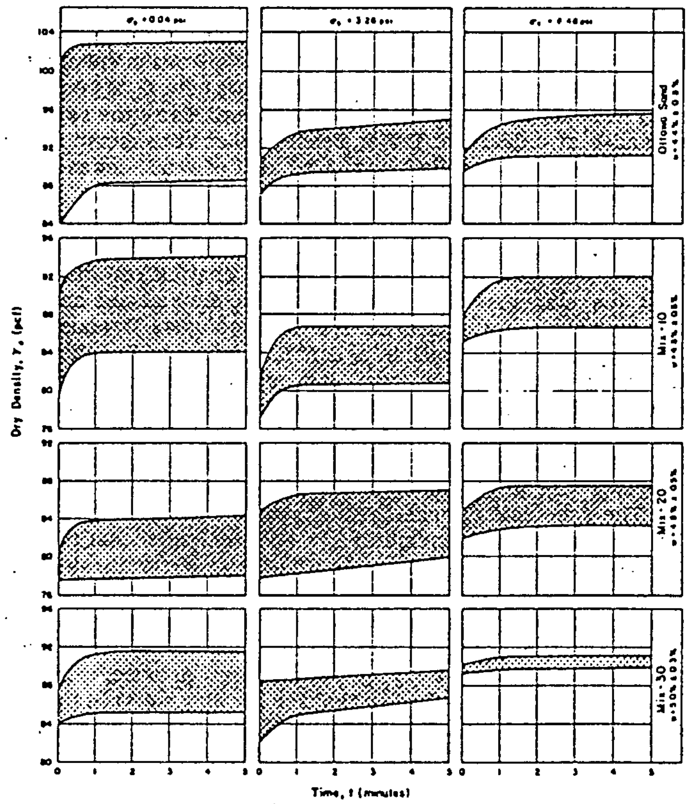


FIGURE 2.29:-TIME RATE OF VIBRATORY DENSIFICATION FOR DAMP SOILS (KRIZEK AND FERNANDEZ,1971)

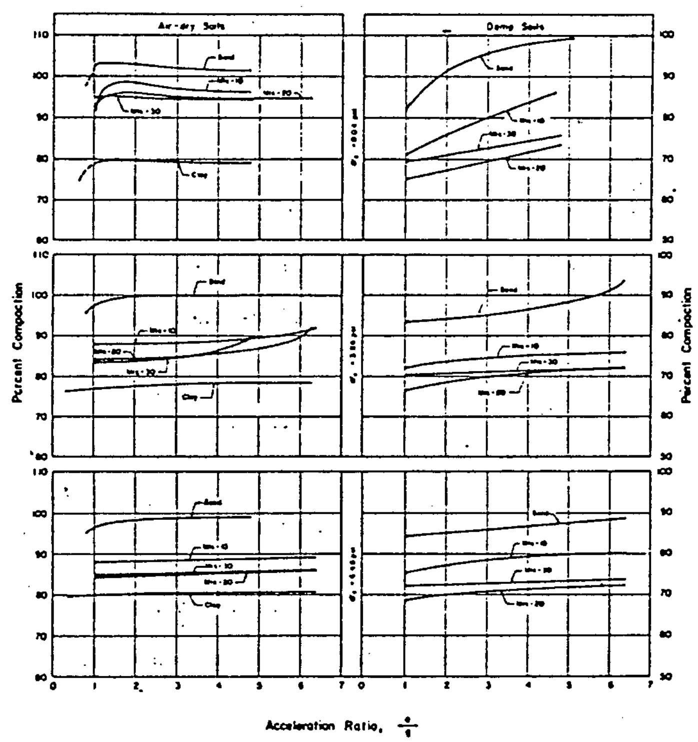


FIGURE 2.30:-PERCENTAGE OF COMPACTION VERSUS ACCELERATION RATIO (KRIZEK AND FERNANDEZ,1971)

> "The concepts apply to sands, gravels, clays, silty soils, base course materials and asphalt concrete. However, the theories are not intended to apply to dry, saturated or submerged, clean sands and gravels, or to saturated silts because these soil conditions involve significantly different mechanisms of vibratory compaction."

The authors commented on four possible mechanisms for explaining the effect of vibration for compacting soils:

i) particle vibration: The application of vibration causes individual soil particles to vibrate leading to rearrangement into either a denser or a looser state. Even a small amount of apparent cohesion such as provided by capillary forces in moist clean sands could restrict densification. Particle vibration was therefore believed to be important only for dry or submerged granular materials.

ii) impact: Impact requires that the roller break contact with the ground surface during each cycle of vibration. The field tests however showed that this usually only happened with vibratory smooth-drum rollers on already compacted material.

iii) strength reduction: For cohesionless soils the possibility existed that the application of vibration reduced the strength of the soil thus facilitating rearrangement of particles. Research on the dynamic properties of soils had shown however, that soils with cohesion generally became stronger under dynamic forces.

iv) cyclic straining: Cyclic deformation of soil produced by oscillation of the roller, provided the best explanation of why roller vibration caused compaction. The mechanism had been demonstrated to be effective in the compaction of soil and worked even with materials with significant cohesion.

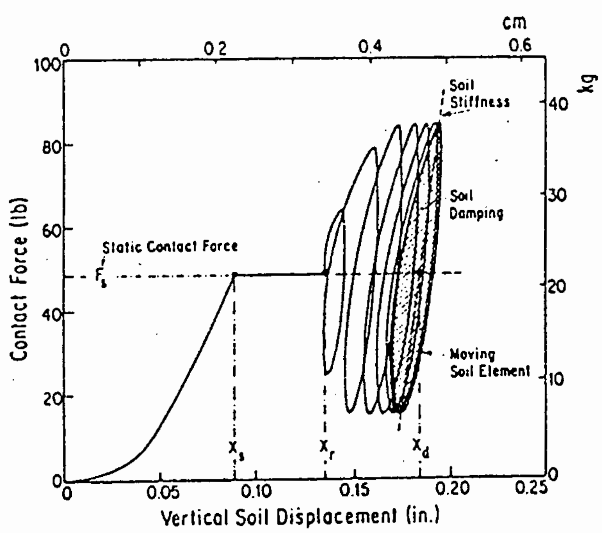
On the basis of laboratory model tests it was concluded that the amount of compaction in the field depended on factors such as roller weight, frame weight, suspension system and the generated dynamic force. The test results suggested that these factors could be represented by the static ground contact force, oscillation per unit distance and the roller vertical displacement.

A two-degree-of-freedom mathematical model was developed. The static contact force and the oscillation per unit of travel distance could be easily controlled but the roller vertical displacement depended on the dynamic response of the mechanical system consisting of the compactor and soil together. Nevertheless the model was verified by full-scale tests.

The discovery that a linear soil model worked well for compaction, which obviously involves significant non-linear, inelastic soil behaviour, was an important finding.

The reason could be explained by the roller vertical force-displacement relationship from the model test. (cf Figure 2.31).

When the roller was lowered onto the soil surface with a contact force Fs it compressed the soil by an amount Xs. During rolling without oscillation the soil compression under Fs was Xr. When the roller oscillated nett compression was Xd. The closed loop represented the soil stiffness and damping felt by the roller. The actual force-deformation behaviour of the stationary soil undergoing compaction is obviously non-linear and highly elastic. However, because the roller remains in contact with the soil during oscillation and is moving forward the soil appears to the roller to have no inelastic characteristics.



2.31

Fig. 9 Typical Vertical Dynamic Force-Displacement Relationship for Model Roller on Soil

3. EXPERIMENTAL WORK.

3.1 Overview.

From the preceding literature survey it is evident that both the type of soil and the compactive technique used have a significant influence on the maximum dry density that can be achieved. Although much work has been done to investigate how parameters such as frequency, amplitude, mould size and surcharge affect laboratory vibratory compaction, the mechanism is still not well understood. It has been conclusively shown however that there exists considerable interaction between the above parameters. (cf Section 2).

Yoo and Selig, (1977) (cf Section 2.7) showed that, despite the intricate mechanism of vibratory roller compaction in the field, it could be modelled mathematically by a simple two-degree-of-freedom model, of the combined influence of the static ground contact force, oscillation per unit distance and roller vertical displacement. It was also pointed out that the combined characteristics of the machine and soil determine the amount of compaction and that the two must therefore be studied together and not in isolation.

The objective of the experimental work was to determine whether Modified AASHTO maximum dry density could be achieved with a graded crushed stone using a vibratory method, and to gain an understanding of the most important factors affecting laboratory vibratory compaction of the material in question.

All tests in this investigation were carried out on a single source of crushed stone from a single quarry stockpile. In addition a filler dust ground from the same quarried material for use as an aggregate filler in asphalt production, was used to provide fines. The characteristics of the material are given in Section 3.2.

Four sets of tests were conducted to examine the effect of altering the grading, frequency, time of vibration and number of layers, mould size and the magnitude of surcharge weight, on maximum dry density.

The test apparatus and basic procedure are described in Section 3.3 and 3.4 respectively, while the actual experiments are detailed in Section 3.5. The results and discussion are presented in Section 3.6 and conclusions drawn from the experiments in Section 3.7.

3.2 Material.

3.2.1 Geological origin.

The crushed stone used was metamorphosed mudrock from the Tygerberg Formation, which is a stratigraphic subdivision of the Malmesbury Group. This material is described in geological terms in UCT Precambrian Research Bulletin No. 15 (Hartnady et al, 1974).

The formation has been lightly folded and has been indurated (i.e. rendered hard by heat) by underlying intrusive granite.

3.2.2 Grading.

The crushed stone* and the filler dust** were used to make up soil samples to fit three grading curves. The three curves are indicated on Fig 3.1. The Talbot equation was used as the basis for deriving the grading, because this equation results in a well-graded soil sample such as is often specified for base course in road construction, (Raston et al, 1976).

$$P = (d/d_{max})^n \times 100$$

P = percentage of the sample smaller than the sieve size m.

d = aperture size of the sieve of size m.

d_{max} = maximum particle size in the sample.

n = an index between 0,5 and 0,3

The range $0.5 < n < 0.3$ is often used by road authorities to define the envelope into which base course gradings must fall.

* Peak Quarry 19 mm base course

** 'Much Asphalt aggregate filler dust' sourced from Peak Quarry.

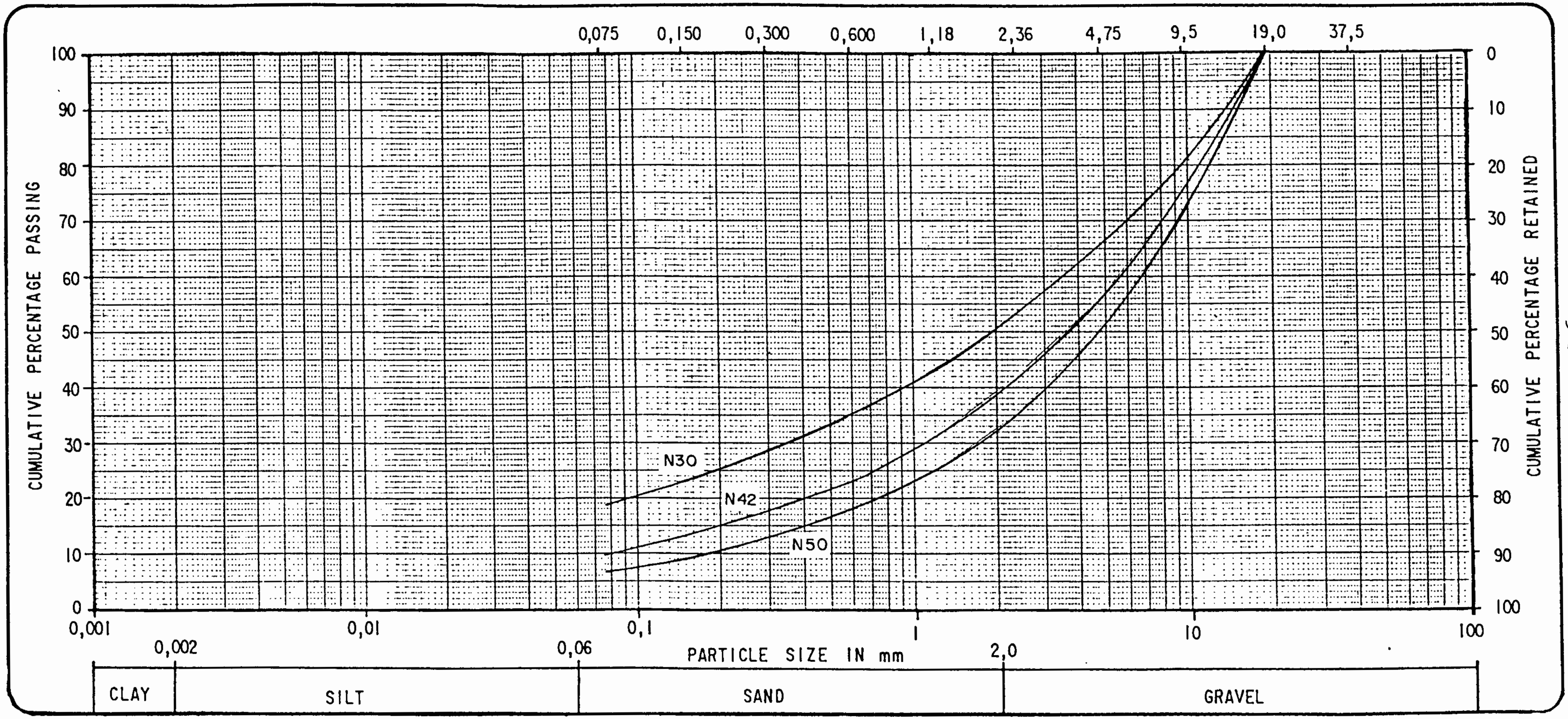


FIGURE 3.1 :- N50, N42 AND N30 GRADINGS USED IN EXPERIMENTS

The three specific gradings were obtained by choosing the power n as 0.50, 0.42 and 0.30 respectively. In this work these three sample types are referred to as N50, N42 and N30 respectively. The specific indices were chosen because they represent the limits of the base course envelope and an intermediate grading near the middle. Data from which Figure 3.1 was prepared is shown in Table 3.1 in the form of the percentage passing each sieve size for each of the three gradings.

Sieve Size (mm)	Percentage passing (d_m)		
	N50	N42	N30
19	100	100	100
4.75	50	56	66
0.425	15	20	32
0.075	6.3	9.8	19.0

Table 3.1 : Gradings for N50, N42 and N30

In order to make up samples to these gradings the following procedure was adopted:-

* The crushed stone was sieved through 19, 4.75 and 0.425mm sieves.

* Material larger than 19mm was discarded.

* Material was sieved into the size fractions indicated in Table 3.2.

* Hydrometer analyses and wet sieving were carried out separately on the crushed rock passing the 0.425mm sieve and on the filler dust, according to Methods A5 and A6 of TMH1 (1979). Of the filler material 97% passed the 0.075mm sieve, while 21% of the crushed rock which passed the 0.425mm sieve also passed the 0.075mm sieve.

* The proportions of each of the sieve fractions required to make up a 6500 gram sample of N50, N42 and N30 were calculated. (cf Table 3.2).

* Samples were made up by weighing out and then mixing the required quantities of each size fraction.

Size fraction (mm)	Mass per sample (grams)		
	N50	N42	N30
19 to 4.75	3250	2860	2210
4.75 to 0.425	2275	2340	2210
0.425 crushed stone	695	813	1014
filler	280	487	1066
	6500	6500	6500

Table 3.2 : Mass of size fractions required for 6500 g sample of N50, N42 and N30

3.2.3 Atterberg limits.

Atterberg limits were determined according to TMH1 Methods A2 to A4 (1979) on the -0.425mm fraction of the crushed stone, the filler and each combination of -0.425mm crushed stone and filler according to the Talbot gradings N50, N42 and N30. The Atterberg limits for each of these materials is given in Table 3.3.

Property	Material				
	-0.425mm	filler	N50	N42	N30
Liquid limit	-	31	20	22	23
Plastic limit	-	26	15	16	17
Plasticity index	Non-plastic	5	5	6	6
Linear shrinkage	1	2	3	2,5	2

Table 3.3 : Atterberg limits

3.2.4 Modified AASHTO maximum dry density and optimum moisture content.

The maximum dry density and optimum moisture content were determined for N50, N42 and N30 materials respectively when compacted under Modified AASHTO compactive effort over a range of moisture contents from 4% to 8%. Standard Test Method A7 of TMH1, (1979) was used. The results of these tests are plotted in Figure 3.3. Maximum dry densities and optimum moisture contents (OMC) for each of the materials is given in Table 3.4.

Grading	Maximum dry density (kg/m³)	OMC(%)
N50	2320	5,0
N42	2335	5,3
N30	2275	5,5

Table 3.4 : Mod AASHTO maximum dry density and OMC

The values in Table 3.4 confirm the conclusions of Machemehl et al, (1972) who, on the basis of their study, concluded that for gradings made up in accordance with the Talbot equation higher densities are achieved for powers of n in the middle of the range (i.e. $0.5 < n < 0.3$) than for those nearer the ends of the range ie $n > 0.3$ and $n < 0.5$.

3.2.5 Dry bulk relative density, apparent relative density and water absorption.

Dry bulk relative mass density, apparent relative mass density (Gs) and water absorption (cf Appendix A.8) were determined for each of the size fractions used to make up gradings N50, N42 and N30 according to test methods B14 and B15 of TMH1 (1979). These properties are presented in Table 3.5. Since many terms for these properties exist and as they are are often confused in the literature, the terms are classified in Appendix A.8.

Property	Size fraction (mm)			
	19 to 4.75	4.75 to 0.425	-0.425	filler
Dry bulk relative mass density	2.721	2.716	2.686	2.686
Apparent relative mass density (Gs)	2.769	2.758	2.686	2.686
% water absorption	0.64	0.57	0	0

Table 3.5 : Dry bulk relative mass density, apparent relative mass density (Gs) and water absorption for size fractions

The bulk relative mass density, apparent relative mass density (Gs) and water absorption for each of the gradings N50, N42 and N30 were calculated on the basis of these parameters for the individual fractions. The properties are tabulated in Table 3.6.

Property	Grading		
	N50	N42	N30
Dry bulk relative mass density	2.714	2.712	2.708
Apparent relative mass density (Gs)	2.752	2.748	2.738
% water absorption	0.51	0.48	0.41

Table 3.6 : Dry bulk relative mass density, apparent relative mass density (Gs) and water absorption of gradings N50, N42 and N30

The increase in apparent relative mass density (Gs) with an increase in fines content is reflected by the values in the table. This is to be expected as the finer fraction includes fewer voids in the material aggregate than the coarse fraction. The percentage water absorption reflects the same property, the percentage being smaller with increased fines content.

3.3 Apparatus.

3.3.1 Vibratory table.

A vibrating table was used in which the vibration was induced by an eccentric-cam, which was driven by an electric motor (single phase, 750 W, 6.2 amp, 220 volt) via a double pulley system. Vibration occurred principly in the vertical plane, although there was some rocking about the pivot axis. When vibrating a soil sample under a surcharge weight it was necessary to steady the weight by hand in order to limit extraneous movements.

The pulley attached to the electric motor rotated at 50 Hz. A frequency of vibration of 50 Hz was achieved by using a 1:1 ratio of driven to driver pulley diameters. Frequencies of 40 Hz and 60 Hz were achieved by using a larger and a smaller driven pulley respectively. Frequencies were measured with a Deumo No 171867 - tachometer.

The amplitude was set and could not be varied by physical adjustment. Under zero load the amplitude was 0.7mm on the pivot axis and approximately 1.2mm on the throw axis. The throw axis is defined as the central axis of the table parallel to the camshaft, while the pivot axis is the central axis at right angles to the pivot axis. The vibrator maintained the amplitude of the table with loads as high as 110 kg provided the surcharge mass was steadied to prevent it "bouncing" out of phase with the vibration. While vibrating a soil sample under surcharge however the amplitude of the table was reduced from 0.70mm initially to 0.35mm after 1 minute as the material was compacted. This is ascribed to interaction of the soil-machine system. The amplitude is discussed in more detail in Appendix B.

3.3.2 The following moulds and surcharge masses were used:

i) A 50kg solid cylindrical surcharge mass with a diameter 150mm, and a series of cylindrical surcharge masses with masses of $\pm$ 5kg each.

ii) A 28kg two-part solid cylindrical surcharge mass with a diameter of 100mm which could be detached into two 14kg masses.

iii) A mould 152mm in diameter, 152mm high, with a detachable collar and a base plate incorporating a central core standing 25mm proud. The base plate was securely bolted to the vibrating table and the mould clamped to the base plate. The effective depth of mould in position was 127mm.

iv) A similar mould and collar to the one described above but 102mm in diameter and with a 200mm effective depth.

3.3.3 Other miscellaneous equipment included:

i) A 300mm ruler calibrated from one end permitting measurement to an accuracy of 0.5mm.

ii) A 16mm diameter round steel tamping rod approximately 450mm long with the ends rounded.

iii) A balance capable of weighing up to 10kg accurate to 5g. (Mettler P10)

iv) A balance capable of weighing up to 2kg accurate to 0.1g. (Sartorius)

v) Two mixing basins approximately 500mm in diameter.

vi) A mixing scoop.

vii) Six flat open containers capable of holding 1000g of material for moisture content determinations.

viii) A drying oven thermostatically controlled and capable of maintaining atemperature between 105°-110°C.

vx) A measuring cylinder with 1000mℓ capacity.

x) Filter paper, 150mm diameter rounds.

xi) A metal measure 150mm in diameter and 180mm high.

xii) Stop watch.

xiii) A steel straight-edge about 300mm in length with one bevelled edge.

3.4 Procedure.

3.4.1 General.

Four groups of tests were carried out in which frequency, mould size, time of vibration, surcharge weight and the number of layers were varied. In all tests however, the basic test, the procedure i.e. the manner in which the material was prepared, the preparation of the mould and preliminary measurements were the same.

3.4.2 Preparation and mixing of the soil sample.

Soil for compaction on the vibratory table was made up to the gradings N50, N42 and N30 in 6500 g samples according to the proportions in Table 3.2.

Before transferring a sample of material to the mixing basin it was weighed to the nearest gram in the air-dried state. The material was thoroughly mixed in the basin whilst still dry, to ensure an even mix of the coarse and fine factions. The amount of water required to bring the material to a predetermined moisture content was added. The material was again thoroughly mixed, covered with a moistened hessian sack and allowed to stand for 10 minutes, so that the moisture could spread evenly through the sample.

After the 10 minute standing period the material was again thoroughly mixed. The metal measure (not the mould) was then filled in two layers. The first layer was scooped into the measure until it was just over half full. The layer was tamped by inserting the 16mm diameter rod to a depth of 30 mm ten times in a regular pattern over the sample. The measure was then filled to the brim and again tamped with the rod ten times. Finally the measure was topped up and struck-off flush with the top. The

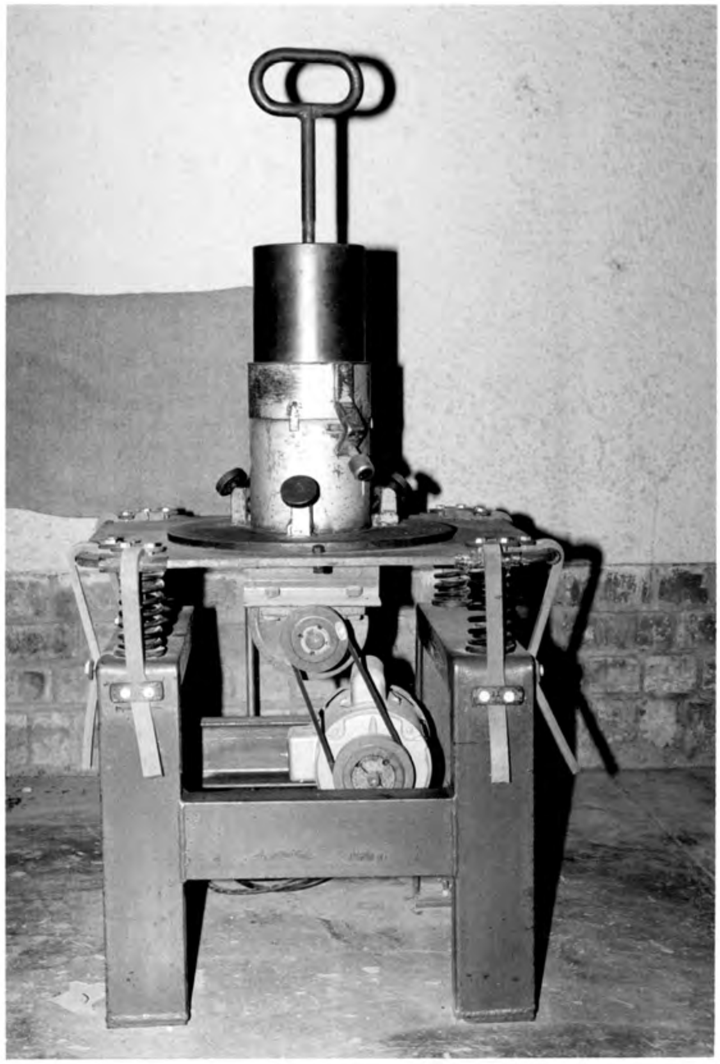


FIG 3.2 EXPERIMENTAL APPARATUS:- VIBRATORY TABLE WITH 152mm MOULD AND COLLAR AND 50 KG SURCHARGE MASS

material determined in this manner was used in the compaction test and was transferred to a clean dry basin and covered with a moist hessian sack. The remaining portion of the sample was set aside for a moisture content determination.

Depending on whether the material was to be compacted in one, two, three or four layers in the mould it was divided up into as many layers in the basin.

3.4.3 Preparation of mould and preliminary measurements.

The base plate was bolted firmly to the vibratory table. First the mould and subsequently the collar were clamped securely in position. Care was taken to ensure that the collar and mould were vertically aligned. A round of filter paper was placed inside the mould to prevent the soil material sticking to the base plate.

The surcharge mass was lowered into the mould, whereupon the height to which the mass protruded above the collar was measured to an accuracy of 0.5mm at three places around the circumference. The average height to which the mass protruded was calculated from these measurements. The mass was removed from the mould, which was then ready to receive the soil.

3.4.4 Compaction procedure.

In most of the tests material was vibrated in two layers. In those instances where material was vibrated in a single layer or in three or four layers, the method was slightly modified as described for the specific tests in Section 3.5.4. The test with two layers was regarded as standard for the purposes of this investigation.

A representative half of the material to be compacted, was scooped into the mould, and as was the case in 3.4.2, tamped ten times with the rod and levelled. The surcharge mass was placed and steadied by hand to keep it vertical.

Except in those tests where the time of vibration was specifically under investigation a layer was vibrated for 2 minutes $\pm$ 10 seconds. For as long as the table vibrated the surcharge mass was steadied by hand without exerting downward pressure in order to prevent excessive rocking of the table.

After the first layer had been vibrated the mass was removed carefully by lifting it slowly whilst, simultaneously twisting it in order to prevent material adhering to it. The other half of the material was placed and rodded ten times before being levelled off. The surcharge mass was again placed in position and the material vibrated for a further 2 minutes $\pm$ 10 seconds.

Once vibration had stopped, but before the surcharge mass was disturbed the extent to which the mass protruded above the collar was measured at three places and averaged. The surcharge mass was removed and the collar unclamped.

The mould with the material still in it was unclamped from the base plate and weighed to the nearest gram. The material was then removed from the mould carefully and the entire sample set aside for moisture content determination.

The mass of the clean dry empty mould was also recorded to the nearest gram.

3.4.5 Determination of moisture content.

Two separate moisture content determinations were undertaken for each compacted sample. The first which is referred to as the "as mixed" moisture content was determined from a representative quantity of 850g$\pm$ of the material left in the basin after the amount for the compaction test had been set aside. The moisture content "after compaction" was determined using the entire compacted sample after it had been pressed out of the mould and the lumps broken up.

The method of moisture content determination was the same in both cases. A sufficiently large container for each sample was weighed accurately to the nearest 0.1 gram. The moist material was placed in the container and both soil and container weighed together. The material was dried in an oven at 105° to 110° over-night, removed from the oven and allowed to cool before being reweighed. All masses were recorded to the nearest 0.1 gram.

3.4.6 Calculations.

The investigation required the calculation of the dry density (γ_d) and the moisture content (w) for each test. The calculation of dry density was facilitated by the computation of a specimen volume factor (SVF).

3.4.6.1 Specimen volume factor.

A specimen volume factor (SVF) was determined as follows:

$$SVF = \frac{4\ 000}{\pi d^2\ (a-b)} \qquad (^1/m^3)$$

where d = diameter of the mould in metres correct to three decimal places.

a = average protrusion of the surcharge weight standing in the empty mould (m)

b = average protrusion of the surcharge weight after the sample has been compacted (m).

The specimen volume factor (SVF) was calculated correct to four decimal places.

3.4.6.2 Dry density

The dry density of each compacted sample was determined as follows:

$$\gamma_d = \frac{W \times SVF}{(w + 100)} \times 100$$

where γ_d = dry density of the compacted sample (kg/m³)

SVF = specimen volume factor (m^{-3})

w = moisture content of the compacted sample (%)

W = mass of the wet material (kg)

$$W = \frac{Mw - Mm}{1\ 000}$$

where Mw = mass of mould and compacted material (grams)

Mm = mass of the clean dry mould (grams)

3.4.6.3 Moisture content

The moisture content in percent was calculated to the nearest 0.1 g as follows:

$$w = \frac{m_1 - m_2}{m_2 - m_3} \quad 100$$

where w = moisture content as a percentage of the mass of dry material

m_1 = mass of the container and wet material (grams)

m_2 = mass of the container and dry material (grams)

m_3 = mass of the clean dry container (grams)

3.5 Experiments.

3.5.1 General.

A total of four groups of tests were carried out on the three gradings N50, N42 and N30. In each group of tests one and in some cases two of the variables were altered in order to assess their effect. The four groups of tests were aimed at investigating the effects of changes in the:

* moisture content and grading
* frequency of vibration
* time of vibration and the number of layers
* surcharge pressure and mould size

3.5.2 The effect of moisture content and grading.

From the literature it was evident that the grading of a graded crushed stone, and the moisture content at which it is compacted, have significant influence on the dry density achieved. Research by the US Bureau of Reclamation has shown that the highest densities are achieved by compacting material in either the oven-dry or saturated condition. Tests by van der Merwe, (1984) on South African crushed stone have also shown that the highest dry density is achieved with moisture contents which, had the water not drained during compaction, would have saturated the compacted sample. Van der Merwe also found however that the shape of the moisture content-dry density curves differed from one material to another as well as from one grading to another. Curves had single peaks, one-and-a-half peaks, two-and-a-half peaks and double peaks.

In order to assess the moisture content-dry density relationship for the crushed stone used in this investigation, each of the gradings N50, N42 and N30 was compacted over a range of moisture contents. The tests were conducted at a frequency of 50 Hz. Material was compacted in two layers in the 150mm diameter mould under a surcharge of 50kg. Each layer was compacted for 2 minutes $\pm$ 10 seconds.

3.5.3 The effect of frequency of vibration.

Having determined the moisture content-dry density relationships for each of the gradings N50, N42 and N30 at a frequency of vibration of 50 Hz, two series of test were carried out on the N50 grading at frequencies of 40 Hz and 60 Hz respectively. All other variables in the test were kept the same as those in the 50 Hz tests.

For the N42 and N30 gradings duplicate samples were compacted at frequencies of vibration of 40 HZ and 60 Hz. These samples were compacted at the moisture content which produced the maximum dry density when compacted at a frequency of 50 Hz.

3.5.4 The effect of time of vibration and the number of layers.

The N50 grading was compacted at a frequency of 50 Hz in the 150mm diameter mould under a 50kg surcharge to assess the effect of the time of vibration on dry density. Four samples were compacted in one, two, three and four layers respectively. The total time for which each layer was vibrated is illustrated in Table 3.7. In the test constituting four layers the material was vibrated for two minutes after each layer was added to the mould. For the test where compaction was carried out in two and three layers the sample was vibrated for 2 minutes after adding each layer. The samples where then vibrated for a further 4 and 2 minutes respectively so that the sample had been vibrated for a total of 8 minutes in each case.

The sample compacted in a single layer was vibrated for a total of 8 minutes. The protrusion of the surcharge mass above the collar was measured at 15 and 30 seconds and 1, 2, 3, 4, 6 and 8 minutes in order to compute the rate at which densification occurred.

Number of layers per sample	Time in minutes for which each layer was compacted			
	Layer 1	Layer 2	Layer 3	Layer 4
1	8	-	-	-
2	6	8	-	-
3	4	6	8	-
4	2	4	6	8

Table 3.7 : Compaction time per layer

3.5.5 The effect of surcharge pressure and mould size.

The N50 grading at the moisture content which gave the maximum dry density in the 150mm diameter mould under a 50kg surcharge when vibrated at 50 Hz, was used for this series of tests.

All samples were compacted in a single layer. Measurements were taken at 15 and 30 seconds and at 1, 2, 4, 6 and 8 minutes to determine the rate of densification with time.

The two cylindrical mould sizes available were used in conjunction with such surcharge loads as could be made up with the various solid masses and loose plates available. Hence samples were compacted in the 150mm diameter mould under surcharge pressures of 42, 27, 20, 17 and 10 kPa. Samples were compacted in the 100mm diameter mould under surcharge pressures of 36 and 17 kPa.

3.6 Results and Discussion.

3.6.1 Moisture Content and Grading.

The results of the tests to determine the moisture content-dry density relationships for the three gradings N50, N42 and N30 are shown graphically in Fig 3.3 and Fig 3.4. In Table 3.8 the maximum dry densities achieved under vibration (MVDD) are compared with the Modified AASHTO maximum dry density (MADD) and the apparent density (AD).

Material	MADD kg/m^3	OMCA %	MADD/AD %	MVDD kg/m^3	OMCV %	MVDD/AD %	(MADD-MVDD)/AD %
N50	2315	4.6	84.1	2346	6.8	85.2	-1.1
N42	2330	5.4	84.8	2315	6.5	84.2	+0.5
N30	2270	5.4	82.9	2227	7.5	81.3	+1.6

Table 3.8 Comparison of MVDD, MADD, AD and MC

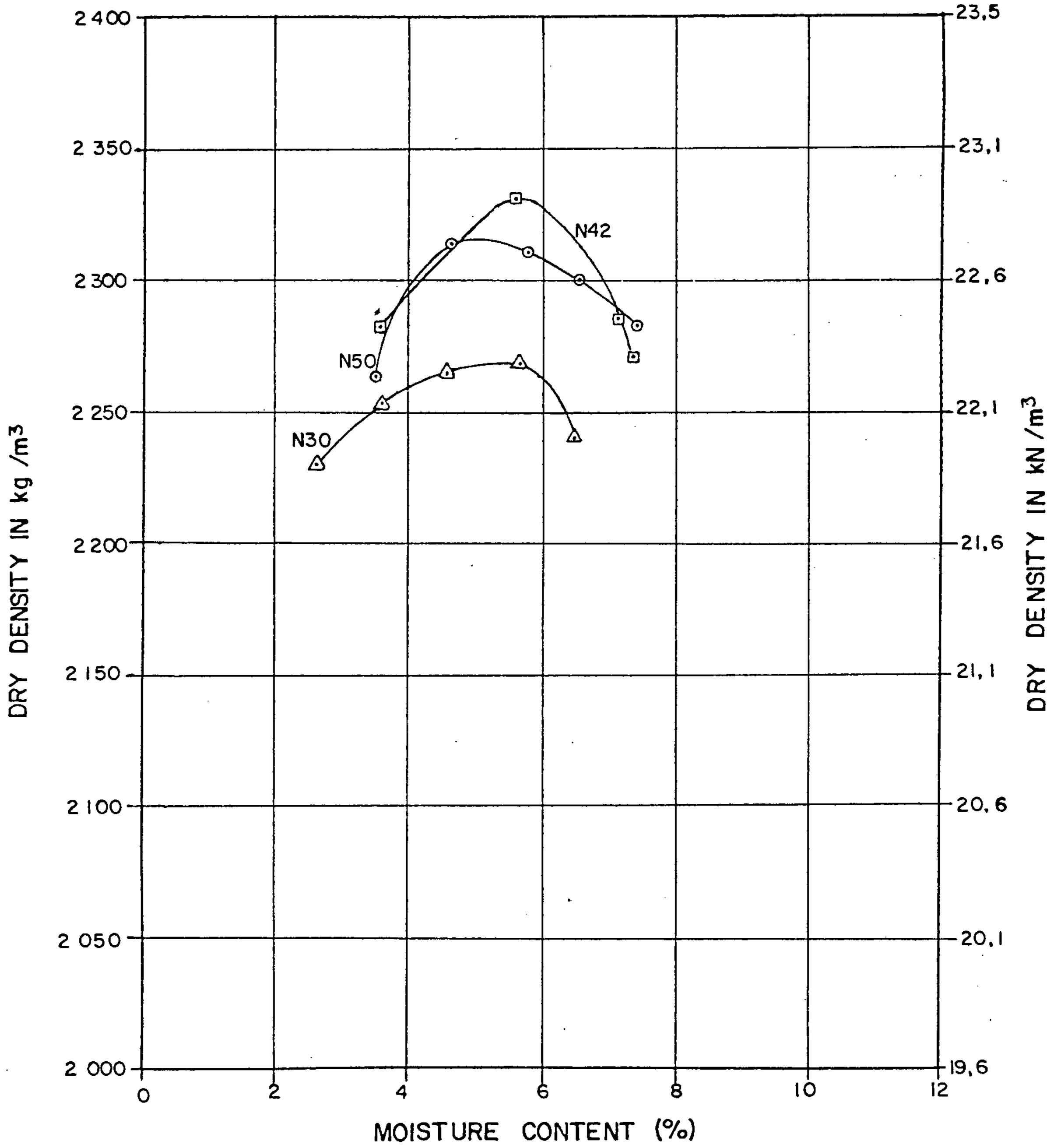


FIGURE 3.3 :- MOISTURE CONTENT / DRY DENSITY CURVES FOR N50, N42 AND N30 COMPACTED WITH MODIFIED AASHTO EFFORT

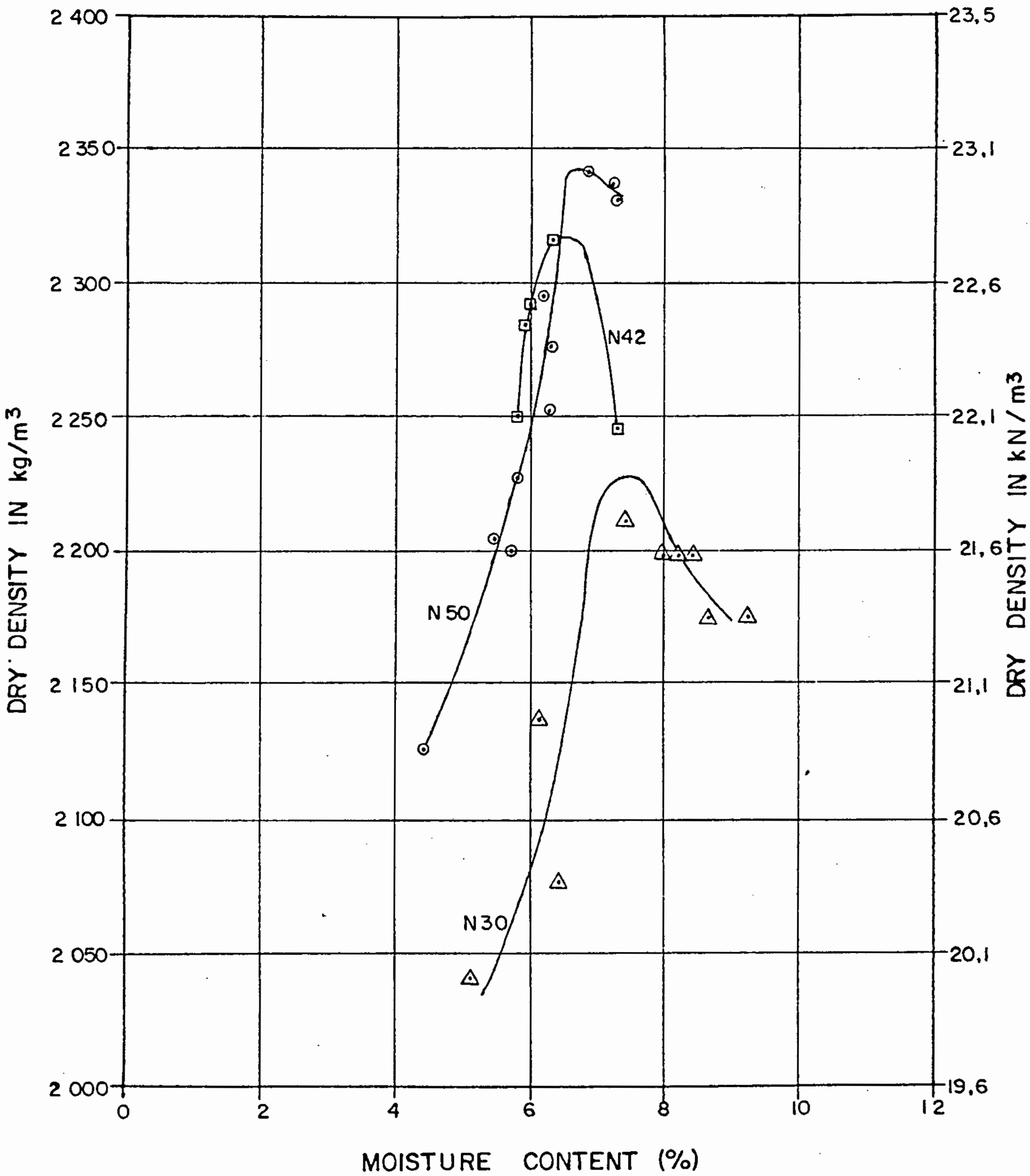


FIGURE 3.4:– MOISTURE CONTENT/DRY DENSITY CURVES FOR N50, N42 AND N30 COMPACTED WITH THE VIBRATORY METHOD

The results in Table 3.8 and Fig 3.7 show that for Modified AASHTO compaction the greatest maximum dry density was achieved with the N42 grading, the lowest with N30 and an intermediate maximum dry density with N50. The dry density achieved with N42 was 2% greater than N30 and 1% greater than N50. Even allowing for experimental error these differences are still significant and confirm the findings of previous researchers, that even small changes in the grading have a significant effect on the maximum dry density. (cf 2.3.3.1)

The N42 contains a percentage of fines intermediate to N50 and N30. The higher dry density achieved with N42 substantiates the explanation advanced by Maddison, (1944) concerning the optimum packing which is illustrated in Fig 2.6 (cf 2.3.3.1)

The optimum moisture contents (OMC) under Modified AASHTO compaction ranged from 4.6% for N50 to 5.4% for both N42 and N30. As expected, marginally more water was required to wet the soil particles of the gradings containing the larger percentage of fine material. (i.e. water demand is higher for soils with more fines).

The results in Fig 3.4 show that under vibratory compaction there is also a maximum dry density (MVDD) and an optimum moisture content (OMCV). In the vibratory method used, the N50 grading yields a greater maximum dry density than N42. N30 again yielded the lowest maximum dry density. This indicates that the optimum proportion of coarse to fine material may not be the same for the vibratory method and the Modified AASHTO compaction.

The optimum moisture contents vary from 6.5% for N42 to 6.8 and 7.5% for a N50 and N30 respectively. In this case the rule that the grading with the greater amount of fines has the higher water demand would appear not to apply.

The OMCV (vibratory compaction) is on average 2% higher than the OMCA (Modified AASHTO). Moreover the drop in dry density above and below the OMC in vibratory compaction is far more pronounced than under Modified AASHTO compaction. This can be seen in Fig 3.5. For a 1% drop in moisture content under vibration there is a 2% drop in apparent density (AD) compared to a 0.5% drop density for the same moisture content differential under Modified AASHTO compaction.

A larger amount of water is therefore needed to facilitate compaction under vibration for the materials in question. The material is also very sensitive to changes in the moisture content, in so far as the peak in the moisture density curve is more pronounced than that of the Modified AASHTO curve. A small deviation in moisture content therefore results in a comparatively large drop in dry density.

The range of 4% of apparent density between the dry densities achieved under vibratory compaction compared to the 2% for Modified AASHTO compaction, also indicates that vibratory compaction is more sensitive to changes in the soil grading.

A study of the diagrams in Fig 3.5 reveals that for N50 the OMCV lies to the right of the zero voids line. This means that the material, as mixed, contained more water than required to saturate the sample in the compacted state. This implies that water was lost during the compaction process by draining from the mould. This was observed to be the case. In order to achieve the maximum dry density with N50 it is necessary to mix in more water than that amount which would result in a saturated compacted sample. From Fig 3.6 it is clear that if 6,4% water is mixed the maximum dry density is achieved, while the moisture content drops to 6.0% during compaction. If however, 6.0% is mixed in to start off with, a substantially lower dry density results. The drainage of the water rather than the amount present during compaction therefore has an important influence on density.

For the N42 and N30 materials no drainage occurred during compaction and as a result the dry densities dropped for moisture contents above the OMCV.

The N42 material contained 10% of minus 0.075mm particles. Dry densities within 1.6% of the Modified AASHTO maximum were still achieved under vibration, despite the soil not being "free-draining".

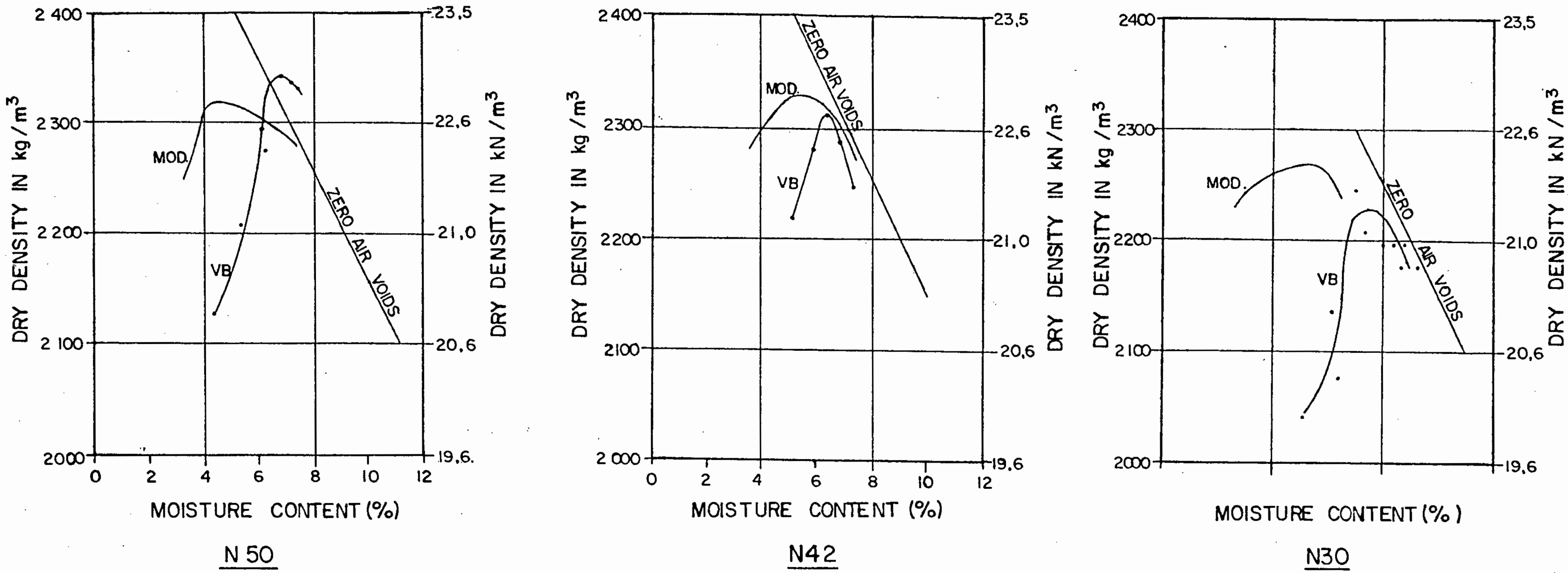


FIGURE 3.5 :- COMPARISON OF MOISTURE CONTENT / DRY DENSITY RELATIONSHIPS OF N50, N42 AND N30 COMPACTED UNDER MODIFIED AASHTO EFFORT AND THE VIBRATORY METHOD.

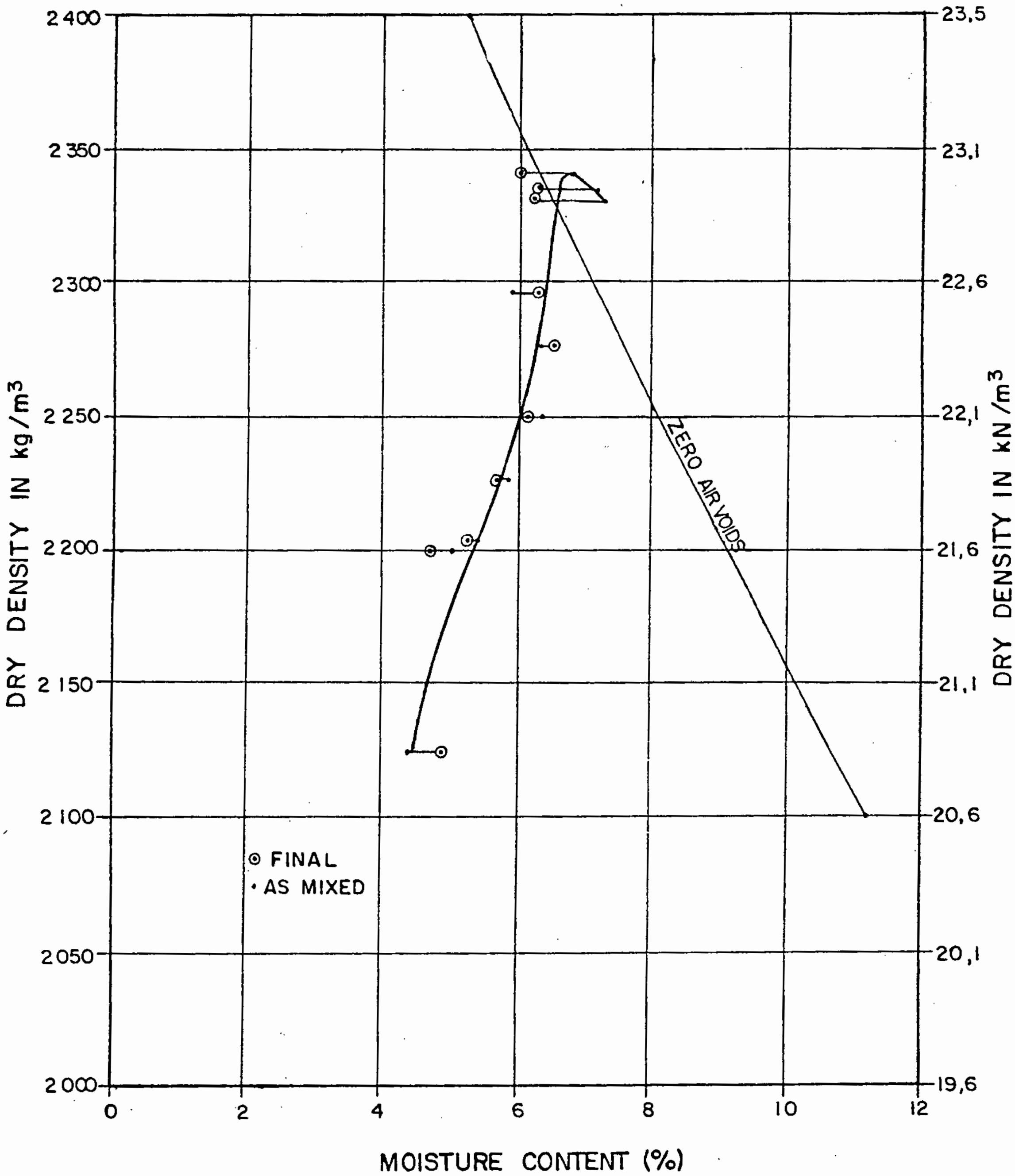


FIGURE 3.6 :- COMPARISON OF "AS MIXED" AND "FINAL" MOISTURE CONTENTS FOR N50 COMPACTED BY THE VIBRATORY METHOD.

Van der Merwe, (1984) stated that for the free-draining crushed stone with non-plastic fines which he tested, the density under the vibratory method was in all cases at least 4% higher than Modified AASHTO maximum dry density, provided the percentage smaller than 0.075mm was less than 12.6%. This would indicate that for the materials in this investigation the plasticity of the fines either inhibited the compaction under vibration or assisted the compaction under impact. Where fewer fines were present as in N50 the vibratory method yielded higher densities but still only by approximately 1% and not 4%.

3.6.2 Frequency.

The results from the tests carried out at different frequencies on N50, N42 and N30 are presented graphically in Fig 3.7.

For N50, which was free-draining, frequencies of 50 Hz and 60 Hz yielded maximum dry densities within 1.2% AD of each other, with 50 Hz producing the marginally higher density. At a frequency of vibration of 40 Hz a significantly lower density was achieved. As the amplitude of the table was not affected by the frequency of vibration for a given surcharge mass the differences in dry density are ascribed to the effect of the altered frequency.

For vibratory roller compaction in the field Yoo and Selig, (1977) found that the frequency of vibration affected only the productivity, as the compaction was due to cyclic straining for which the number of cycles rather than the rate was significant. Whereas it is not suggested that what applies in the field, applies necessarily also in the laboratory, if Yoo and Selig's findings were extended to these tests, one might expect that N50 compacted at 40 Hz would reach the level of density achieved with N50 and 60 Hz, given enough time.

It was found that the N50 could hold a maximum of 7.8% water prior to compaction, so that it was not possible to achieve a saturated sample after compaction. The apparent increase of density with moisture content at 40 Hz appeared moreover to be due rather to a more advantageous packing of particles as a result of wetting, than to densification under vibration. The surcharge weight was not seen to settle under vibration. However, even if the moisture-content dry density curve is extrapolated, a density less than that at 50 Hz or 60 Hz will be achieved at the zero voids line.

Forssblad, (1981) found that for a given amplitude of vibration there existed an optimum frequency for field compaction. There may be a similar optimum under laboratory conditions. For N50 this frequency would appear to be in the region of 50-60 Hz. It is also possible that there is a minimum threshold frequency which in this case would lie above 40 Hz.

The results with N42 show a similar trend to those with N50. The dry density at 60 Hz however is marginally higher than with 50 Hz. This may be because the optimum frequency for N42 lies nearer to 60 Hz than to 50 Hz. At 60 Hz the excess water was expelled during compaction as was the case with N50, whereas at 50 Hz the material held the water and a lower density resulted.

The densities with N42 at 40 Hz are, as with N50 significantly lower than those achieved with 50 Hz and 60 Hz.

With N30 which contains 19% of minus 0.075mm size particles, the effect of the frequency of vibration was not nearly as pronounced as for N50 and N42. On average the densities at 50 Hz and 60 Hz achieved with N50 and N42 were 3.5% of AD higher than with N30, whereas at 40 Hz the density with N30 is in fact from 1 to 2% of AD higher than either N50 or N42.

For the two samples tested at 40 Hz the dry densities differ by 2.6% of AD whereas those at 60 Hz are effectively similar. The densities achieved at 40 Hz may be more erratic than those at 60 Hz because at 40 Hz the effect of other factors affecting compaction are more pronounced.

In the standard test method - ASTM: D4253-83 frequencies of 50 or 60 Hz may be used. The use of one or other of these frequencies is not specified for any particular grading but either is permitted provided the amplitude and time of vibration are chosen accordingly. The amplitude is chosen in such a way that the peak acceleration (a_{max}) is a constant regardless of which frequency is used. The test results shown in Fig 3.7, suggest that an increase in peak acceleration such as is achieved by increasing the frequency while keeping the amplitude constant, does not necessarily increase the compacted density.

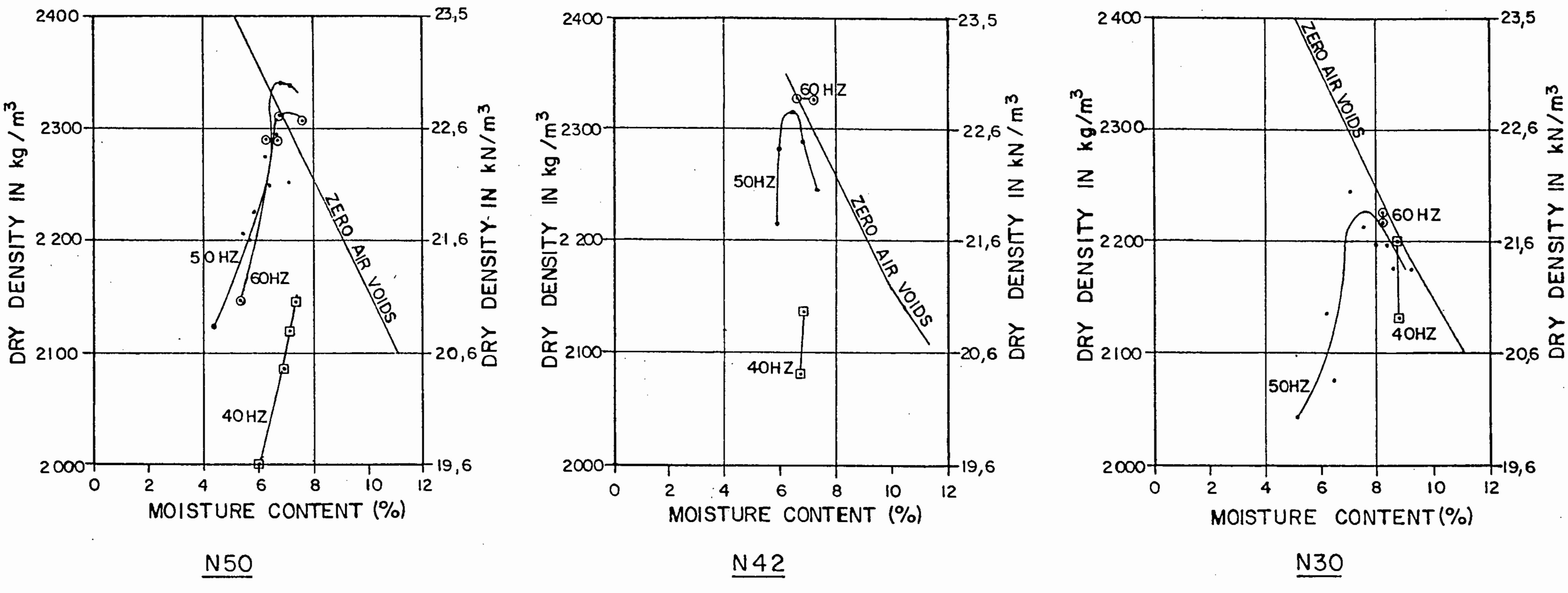


FIGURE 3.7:- MOISTURE DENSITY RELATIONSHIPS FOR FREQUENCIES OF 40HZ, 50HZ AND 60HZ

3.6.3 Time of vibration and compaction in layers.

The rate of increase of dry density, with time, is illustrated in Fig 3.8 and 3.9 for the N50 sample compacted in a single layer. The time of vibration is plotted to a natural scale in Fig 3.8 and to a log scale in Fig 3.9. When plotted to a natural scale it is clear that more than 98% of the density at 8 minutes has been achieved after 4 minutes. The density at 8 minutes was 85.3% AD. When plotted to a log scale, however, the density can be seen to be increasing steadily up to 6 minutes before the curve starts to flatten. Had the tests been continued to 10 or 12 minutes a further increase of 2% of AD above that realized at 8 minutes may well have been achieved.

For those samples compacted in two, three and four layers the density achieved was plotted against the average time of vibration per unit volume. These times were 5,6 and 7 minutes for compaction in 4, 3 and 2 layers respectively. The results indicate that regardless of the number of layers in which the sample was compacted the dry densities achieved after a total of 8 minutes of vibration were within 0.8 % of AD of each other. As the average time of vibration per unit volume was the lowest for 4 layers while the density achieved was the highest, this could be viewed as the most efficient method of compaction.

ASTM: D4253-83 on the other hand specifies compaction in a single layer. The method also requires compaction for a minimum of 10 minutes at 50 Hz. It is possible that at 10 minutes the curves converge. Further testing for a longer time would be necessary to verify this.

During vibration N42 and N30 samples which contained the greater percentages of minus 0.075mm particles, the fines appeared to segregate from the sample in the form of a thick slush. Although no tests were conducted for more than 4 minutes, with these two gradings, it is felt on the basis of observation that, for periods of vibration of 8 minutes and more, these gradings would be significantly altered through loss of fines. For N42 and N30 gradings compaction in 3 or 4 layers is therefore recommended. For free-draining material with few fines such as N50, compaction in a single layer for 10 minutes as recommended in ASTM: D4253-83 will probably yield as high a maximum dry density as compaction in more layers.

In the field, of course, layers of 150mm and more are compacted without fear of segregation. The loss of fines in the the form of slush from the gradings with the larger percentages of fines can be viewed as an expulsion of excess fines leading to higher density, albeit then with a different grading.

3.6.4 Mould size and surcharge pressure.

The result of the tests to assess the effect of mould size and surcharge pressure on density are shown graphically on Figs 3.10 to 3.13. Density is plotted against time on a log scale.

All the tests were conducted with N50 which was shown to be free-draining under vibration. The 5% of -0.075mm particles in N50 had a plasticity index of 5. From the tests to establish the effect of moisture content and grading on density, N50 was found to behave as a free-draining cohesionless soil. The ASTM: D4253-83 test method would therefore be applicable to this material. It is important to note that the ASTM: D4253-83 method specifies the frequency, amplitude and mould size used. A pressure of 14 kPa in a 150 mm diameter mould is also specified and requires a mass of 26 kg.

From Fig 3.10 it is immediately apparent that after 8 minutes of vibration the density achieved under a surcharge of 50 kg, which is approximately 27 kPa, is substantially higher than that achieved with any of the other surcharge pressures. The literature revealed that amplitude was deemed to have a significant effect on the efficiency of compaction under vibration. Measurements of the amplitude of the table under various surcharge masses showed the amplitude was not affected by the surcharge mass for the equipment used. The lower density achieved under 38 kPa (70 kg) surcharge pressure cannot therefore be ascribed to reduction in amplitude of vibration due to the increased total mass on the table.

There evidently exists an interplay between the surcharge mass and the material vibrated quite independent of the amplitude of vibration of the table.

Fig 3.11 illustrates the densities achieved in the 100mm diameter mould under surcharge pressure of 36 kPa and 17 kPa. The difference between the densities is 1.7% of AD. From the two results it is not possible to determine whether an intermediate surcharge pressure would have yielded a dry density significantly higher than those under 36 kPa and 17 kPa. Bearing in mind that with the 150mm diameter mould it was neither the 17 kPa nor the 38 kPa but the intermediate 28 kPa which yielded the highest density by a clear 6% of AD, the same could be expected if a 28 kPa pressure was applied with the 100mm diameter mould.

The foregoing statement suggests that the efficiency of compaction in the test depends for a given amplitude and frequency on the surcharge pressure only. Fig 3.12 shows the densities achieved in the two size moulds under similar surcharge pressures. Curves are plotted for 38 kPa and 17 kPa pressures. The difference in density under 38 kPa for the two mould sizes is only 0.5% of AD while for 17 kPa the difference is a more significant 2.4% of AD. Neither of these pressures is necessarily the optimum and therefore the results are inconclusive. In order to assess whether for constant amplitude, frequency and surcharge pressure the mould size affected the density a test would have to be carried out where the N50 is compacted under 28 kPa in the 100mm diameter mould. This density should then be compared with that under similar pressure in the 150mm diameter mould.

In ASTM: D4253-83 different size moulds are specified depending on the maximum particle size. This suggests that the compilers of this test regard the mould size as having a significant effect. Pisarczyk, (1980) pointed out that provided the diameter of the mould is at least 5 times the maximum particle size the mould size has no significant effect. The results as presented on Fig 3.13 appear to support this view.

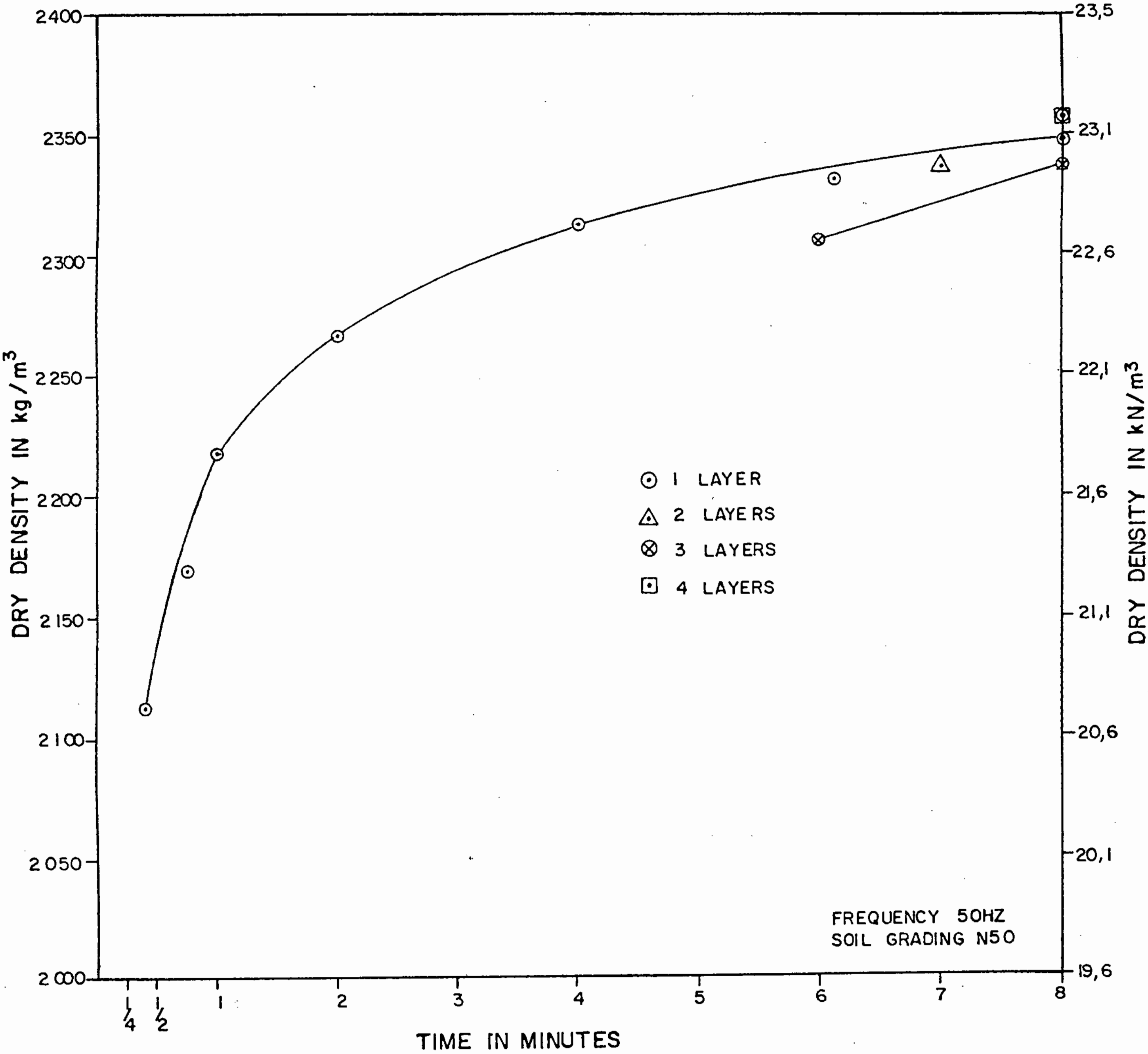


FIGURE 3.8 :- DRY DENSITY VERSUS TIME OF VIBRATION TO A NATURAL SCALE

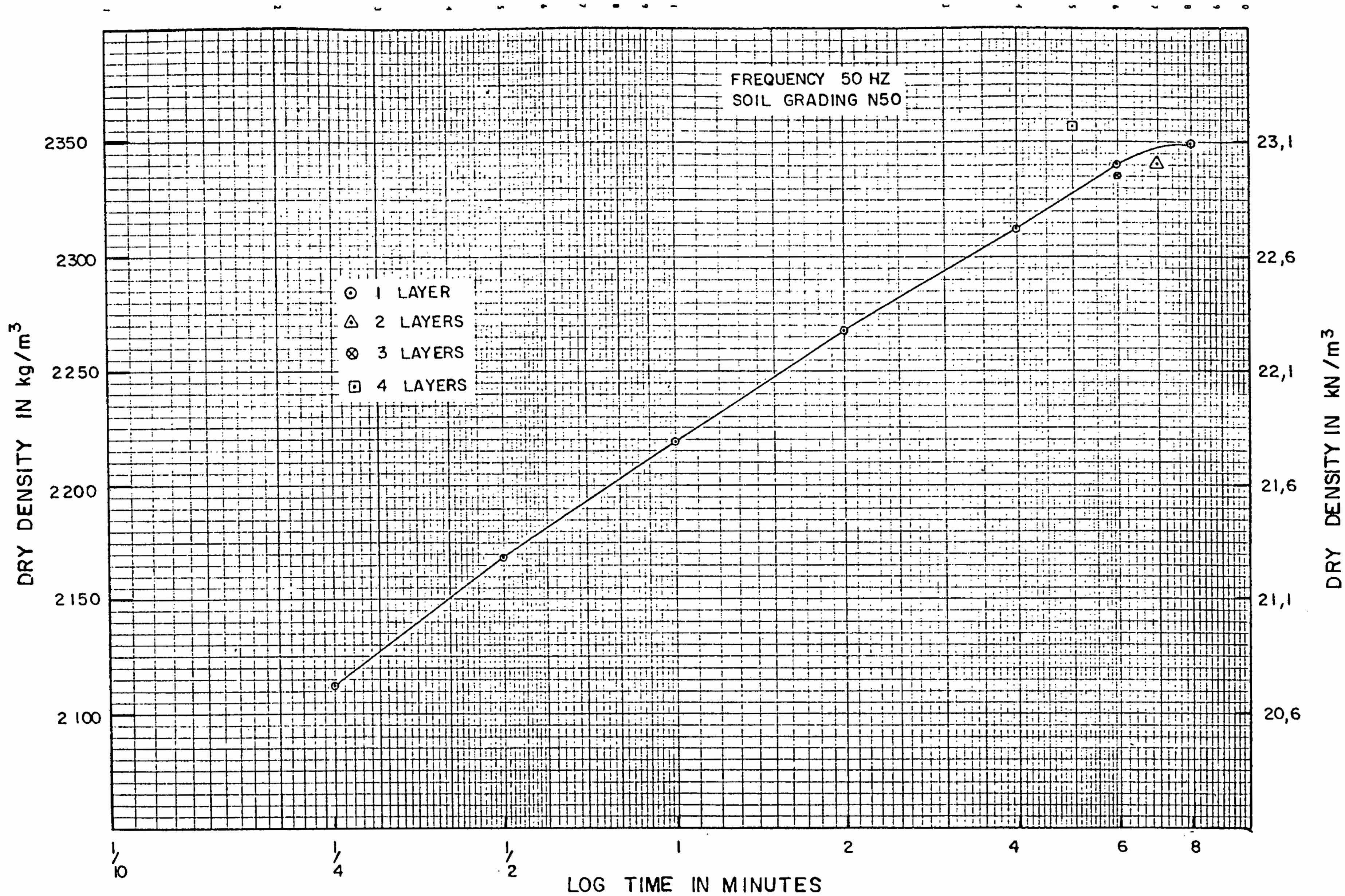


FIGURE 3.9:- DRY DENSITY VERSUS TIME OF VIBRATION TO LOG SCALE

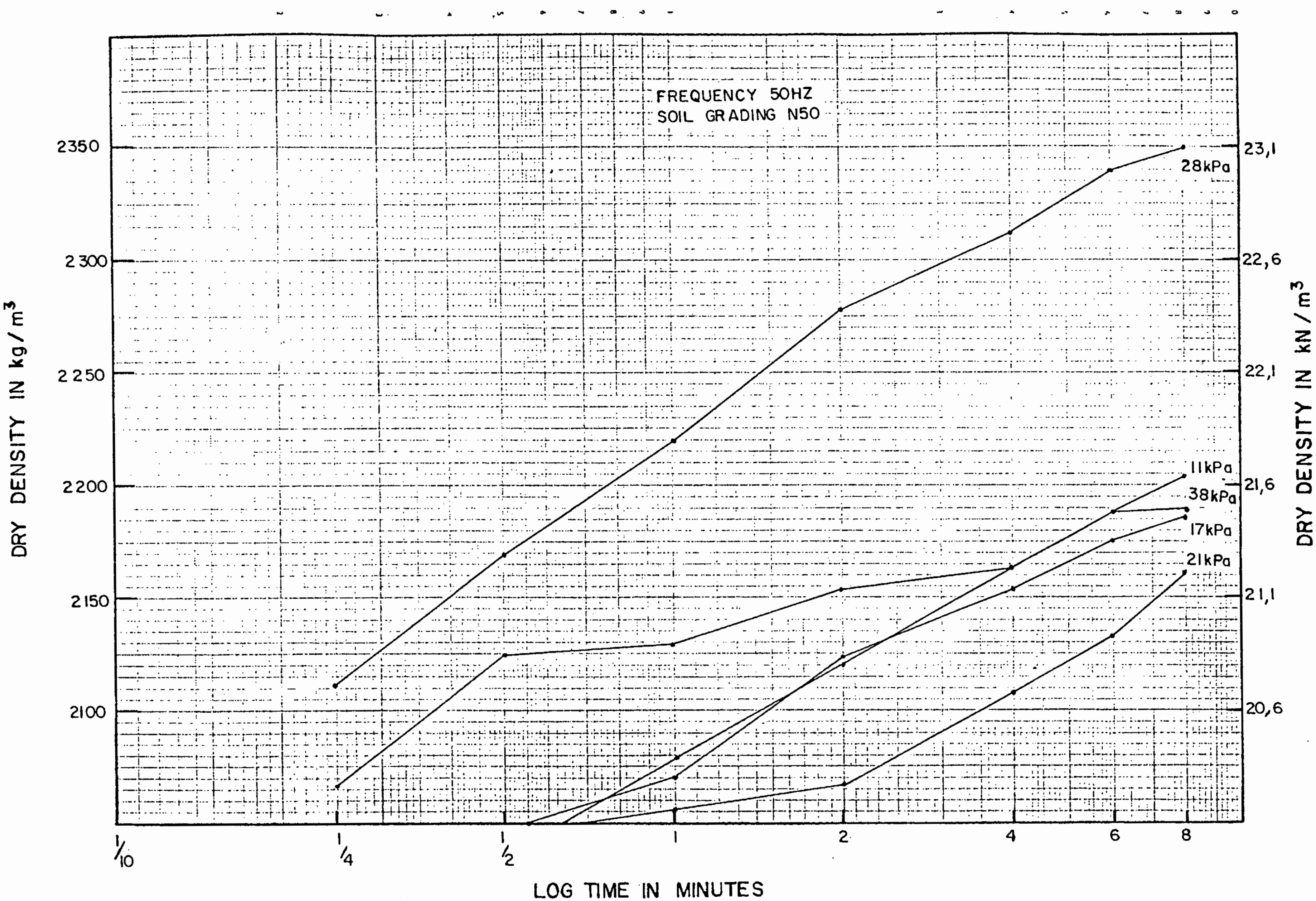


FIGURE 3.10 :- DRY DENSITY VERSUS TIME OF VIBRATION WITH VARYING SURCHARGE AND A 150mm DIAMETER MOULD

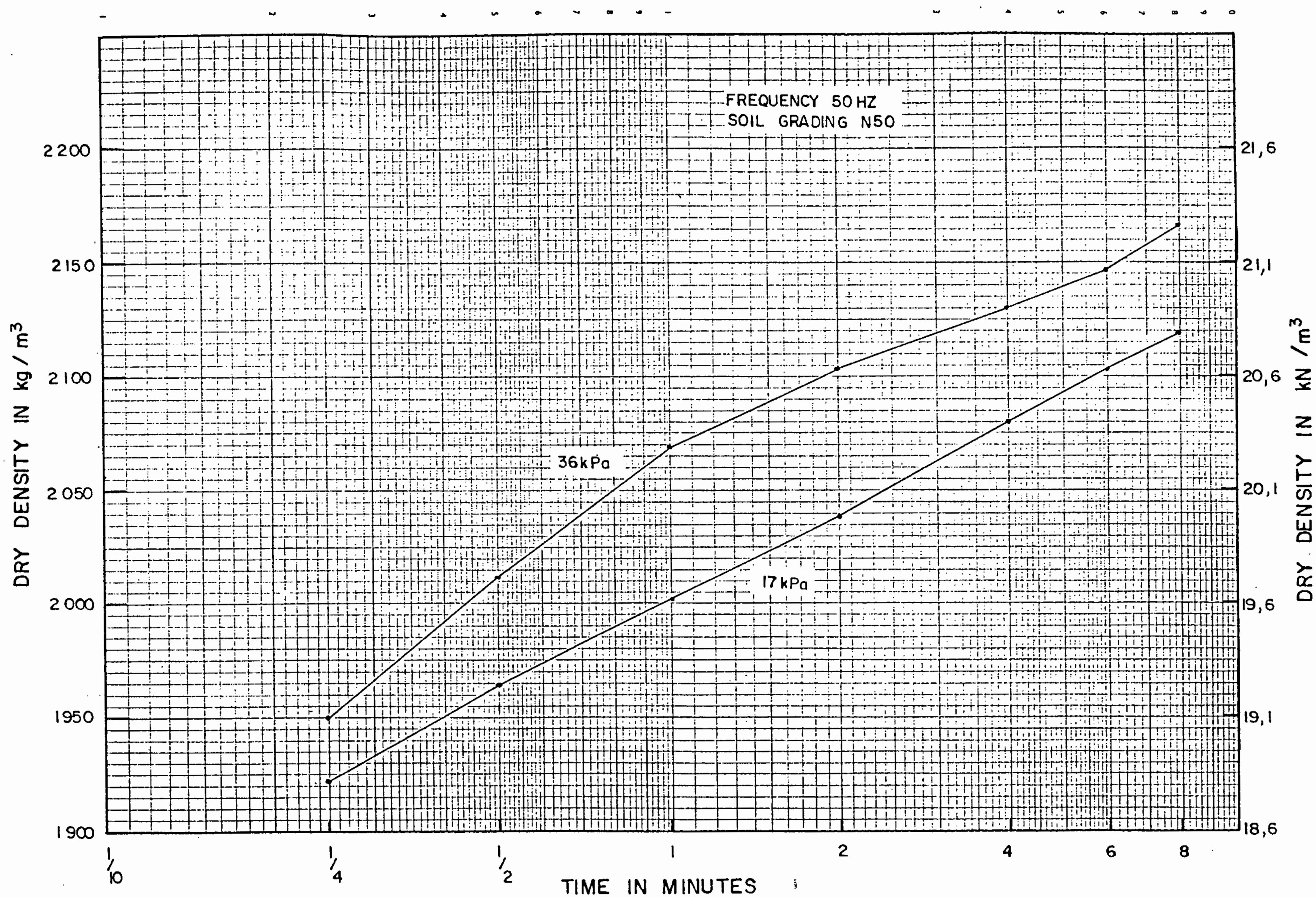


FIGURE 3.11 :- DRY DENSITY VERSUS TIME OF VIBRATION WITH VARYING SURCHARGE AND A 100mm DIAMETER MOULD.

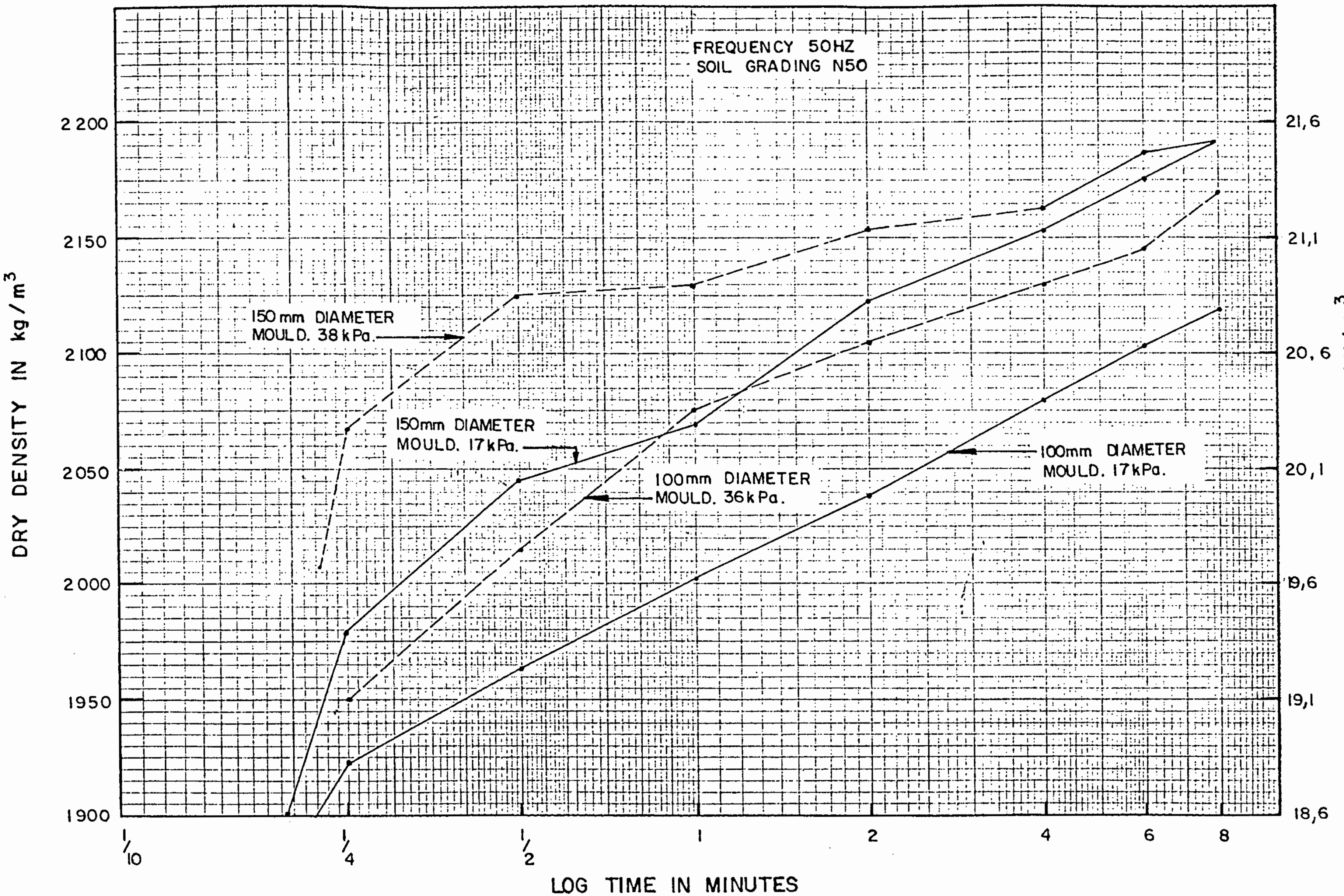


FIGURE 3.12:- COMPARISON OF DRY DENSITY VERSUS TIME CURVES FOR COMPACTION WITH CONSTANT SURCHARGE PRESSURE.

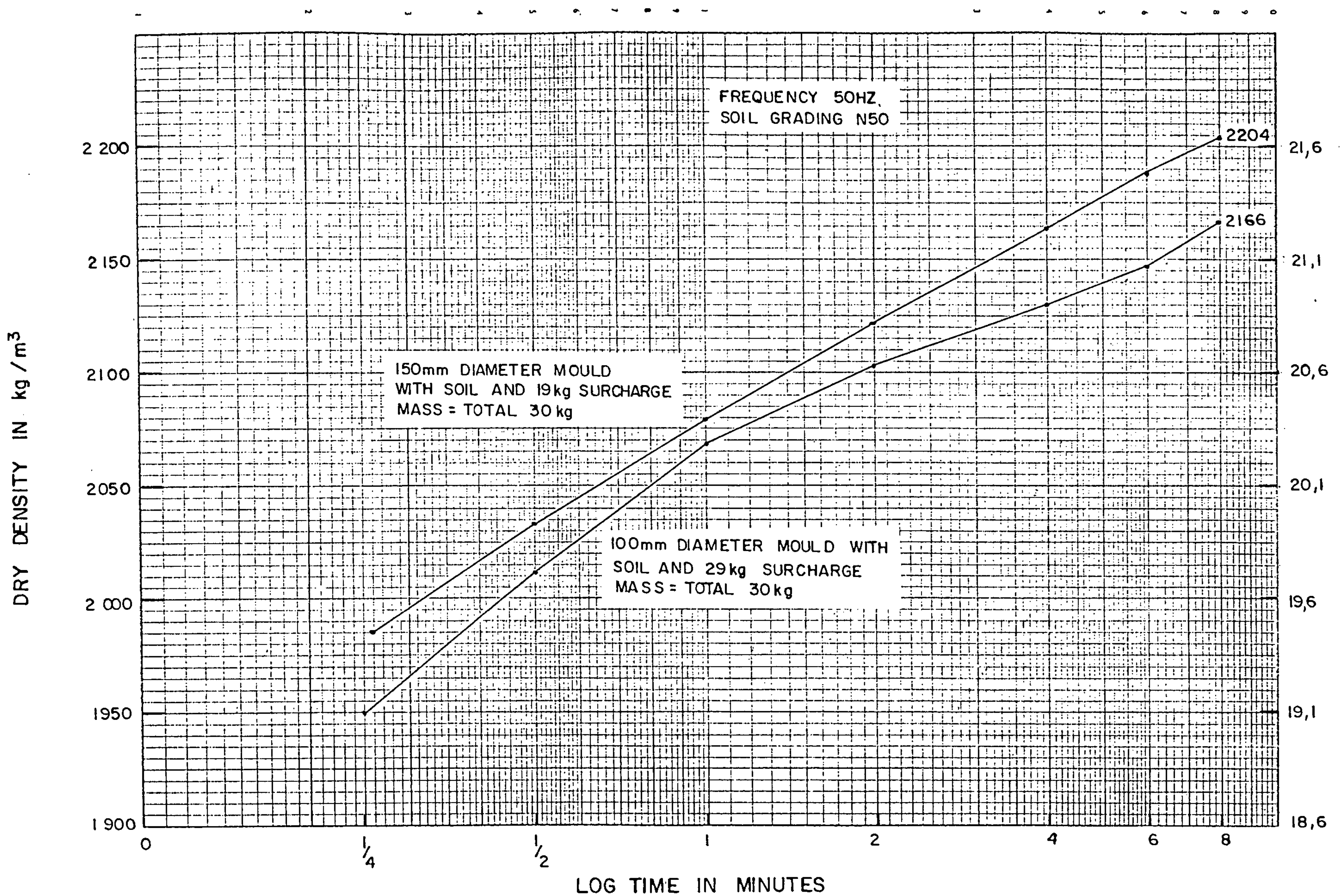


FIGURE 3.13 :- COMPARISON OF DRY DENSITY VERSUS TIME FOR COMPACTION WITH CONSTANT TOTAL MASS ON THE TABLE.

3.7 Conclusions on experimental work.

i) All three of the gradings, N50, N42 and N30, were compacted to 83-85% of apparent density under modified AASHTO compactive effort. Maximum dry densities of 84-85% of apparent density were achieved with N42 and N50 under vibration, while with N30 a maximum dry density of 81% of apparent density was achieved. For the graded crushed stone in question the vibratory test yielded dry densities of the order of modified AASHTO maximum dry density.

ii) The optimum moisture content under modified AASHTO was $\pm$ 5% compared to $\pm$ 7% under the vibratory method. The dry density achievable under the vibratory method was found to be is more sensitive to changes in moisture content than under modified AASHTO compaction. For the N50 which was free-draining under vibration, the OMC was that moisture content at which the uncompacted sample was saturated (i.e. sufficient water to exclude all air in the voids prior to compaction). For N42 and N30 the OMC was such that sufficient water was present to just saturate the sample prior to compaction, without excess moisture, as such,excess moisture could not be expelled from the soil during vibration, due to the poor permeability of these gradings.

iii) The frequency of vibration has a significant impact on the dry density. For N50 and N42 compaction was limited at 40 Hz, whilst densities of the order of modified AASHTO maximum dry density were achieved at 50 Hz and 60 Hz. At 40 Hz the compaction of N30 was marginally less and more erratic than at 50 Hz and 60 Hz. For the crushed stone in question, the N50 and N42 gradings require a minimum frequency of $\pm$ 50 Hz for effective compaction. As the N50 compacted best at 50 Hz and N42 at 60 Hz, there appears to be an optimum frequency, which is dependent of the grading. For the N30 grading which contained more fines the frequency appears to have less influence on dry density.

iv) The minimum time of vibration should be 10 minutes when compacting soil in a single 127mm thick layer. If, because of the nature of the material segregation is likely to occur, compaction of a sample should be done in 3 or 4 layers with 2 minutes of compaction per layer and a minimum of 10 minutes in total. The number of layers does not, however, affect the dry density, where segregation is not a problem.

v) Provided the mould is cylindrical and at least five times the maximum particle size in diameter and depth, the mould size has no significant effect on dry density.

vi) Under the specific test conditions there existed an optimum deadweight surcharge pressure of 27 kPa with the 150mm diameter mould.

vii) There is considerable interaction between such variables as surcharge, amplitude, frequency and time of vibration.

4. OVERALL CONCLUSIONS.

4.1 In the field, vibratory compaction was introduced in the 1930's. Until 1960 the application was limited to the compaction of cohesionless soil, because the compaction achieved by non-vibratory methods with cohesive soils was considered adequate at the time. Since 1960 vibratory compaction has been found to be effective and therefore utilized to an ever-increasing extent on all types of soil. Today vibration is used to compact the full range of soils including rockfill, soil cement, sand, crushed stone, silt, clay and even asphalt.

4.2 Standard laboratory tests utilizing vibratory compaction have been developed in the USA, Europe and also South Africa. These are applicable specifically to cohesionless, free-draining material. In the US and South Africa the standard test methods specify vibration on a vibratory table under surcharge, while in Europe compaction is by vibratory tamper. The mechanism of vibratory compaction in the laboratory appears not to be understood properly. This is because, although the principal factors have been identified, the interaction between these factors needs to be investigated further. (ASTM Designation: D4253-83 is the most recently published standard test).

4.3 The application of laboratory vibratory test procedures for the determination of consistent maximum index densities for soils appears to have definite merit. However, the interaction of the principal variables affecting compaction by vibration in a mould are, at this stage, not sufficiently well understood to permit the prescription of a single test method, applicable to all soil-types.

4.4 Soils for which laboratory vibratory compaction may be used with confidence are coarse-grained soils in which the minus 0.075mm fraction does not exceed 12%. In addition, the plasticity index of the minus 0.425mm fraction should not exceed 4.

It is recommended that the ASTM Designation D 4253-83 should be used for these soils.

This test method recognizes that the variables frequency, amplitude, mould size and time of vibration interact and provides for calibration in order to determine the optimum combination of these factors for a given soil.

The ASTM method specifies a constant 14 kPa surcharge pressure. This is seen as a significant shortcoming of the ASTM test method, as both the literature survey and the experimental work described in Chapter 3 indicate that there is a significant interaction between soil type, amplitude and surcharge pressure.

It is therefore recommended that the calibration should be extended to investigate an optimum surcharge pressure in the range 14 to 38 kPa, and that the surcharge mass should be in a single solid block.

A modified ASTM 4253-83 should include:

frequency	:	50 or 60 Hz (whichever is available)
amplitude*	:	variable from 0,05 mm to 0,64 mm
surcharge*	:	in the range 14 to 38 kPa
mould size	:	cylindrical, with the diameter a minimum of five times the maximum particle size
layer	:	soil compacted in one or more layers, depending on the tendency of the material to segregate during compaction
time	:	vibrate for a total of 12 minutes
moisture content	:	as wet as possible but no free water

4.5 There is sufficient evidence in the research reviewed in the literature survey to suggest that a laboratory test for cohesive soils can also be developed. Before this can be done, however, the interaction between amplitude, surcharge pressure and soil-type will have to be studied. Whereas the vibratory table and surcharge configuration specified in ASTM D 4253-83 appear to work for cohesionless soil, the European approach of clamping the mould to a fixed base and vibrating the surcharge in a controlled manner is likely to be more appropriate for cohesive soils, since these require positive displacement to achieve compaction.

* optimum to achieve maximum dry density to be determined by calibration.

5. RECOMMENDATION FOR FURTHER WORK.

Further research aimed at understanding the mechanism of vibratory compaction in the laboratory, whether the soils are cohesionless and free-draining or not, should endeavour to assess the interaction of the amplitude and surcharge pressure for a given soil.

As considerable research has already been done in the US using a vibratory table it is deemed best to persevere with this system in the first instance, rather than the method using a vibratory tamper preferred in Europe, for which less data is available. The vibratory table should be so designed that the vibrating oscillation is in the vertical plane only and any extraneous movements are prevented. The table should be of such a size and mass that the vibratory characteristic remains unaffected by the total mass on it up to $\pm$ 200 kg.

A series of tests should then be carried out in which a soil type is compacted at different amplitudes, such that for each amplitude a number of samples are compacted under different surcharge masses. During testing the amplitude of both the table and the surcharge mass should be continuously monitored.

It is suggested that the N42 grading used in the experiments described in Section 3 be used, together with the 152 mm diameter mould. For the above series of tests the frequency of vibration should be set at 50 Hz, as compaction below this frequency has been found to be unreliable. Soil should be compacted in a single layer despite possible segregation for a period of 12 minutes.

Depending on the results, consideration should be given to applying the surcharge pressure by means of a spring or by means of a system where the mould is fixed and vibration imported from above. (cf Pisarczyk, 1980 and Shklarsky, 1980).

Any surcharge mass used should vibrate as a single mass and not as a series of plates which are likely to vibrate out of phase, thus introducing an unnecessary variable.

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APPENDIX A - Explanatory notes and definitions

A.1 Talbot equation. (Roston et al, 1976)

$$P = (d/d_{max})^n \times 100$$

where P = percentage passing a sieve with opening d mm

d_{max} = maximum stone size in the sample in mm

n = an index (usually $0.50 < n < 0.30$)

Grading curves which fit the Talbot equation are well-graded. The equation is commonly used to derive a grading envelope for graded crushed stone for basecourse.

A.2 Uniformity coefficient.

$$C_u = (d_{60}/d_{10})$$

where d_{60} and d_{10} = the particle size at 60% and 10% of the cumulative per cent passing particle size distribution curve.

The uniformity coefficient is normally used to indicate how uniform rather than how well-graded a sand is. The smaller the ratio, the more uniform the sand.

A.3 Amplitude.

The amplitude of vibration in the text is defined in Fig A.1.

amplitude = A

double amplitude = 2A

A.4 Peak acceleration.

The peak acceleration (a_{max}) referred to by Dobry and Whitman, (1972) and others in the text is defined as follows.

$$a_{max} = (2\pi f)^2 \times A \text{ in } {}^m/s^2 \quad \text{or} \quad a_{max} = (2\pi f)^2 \times (A/g)$$

Where f = frequency

A = amplitude

g = acceleration due to gravity (i.e. 9.81 $^m/s^2$)

A.5 Fines.

Fines are defined as that portion of a soil finer than a No. 200 US standard sieve i.e. smaller than 0.075mm.

A.6 Optimum moisture content (OMC).

The optimum moisture content is defined as follows:

i) The water content at which a soil can be compacted to a maximum dry unit weight by a given compactive effort. (ASTM Designation D653-86).

ii) The optimum moisture content for a specific compactive effort is the moisture content at which the maximum density is obtained. (TMH1, Method A7).

The optimum moisture content is therefore not an absolute value for a given soil but is a function of the compactive technique.

A.7 Saturation.

The voids of a soil may be filled with air or water or both. If only air is present in the voids the soil is dry, whereas if only water is present the soil is saturated. The soil is said to be partially saturated if both air and water are present. The degree of saturation (S_r) is defined as:

$$S_r = \frac{\text{Volume of water}}{\text{Volume of voids}}$$

The degree of saturation is therefore a function of the volume of voids in a soil. For a soil which is being compacted the degree of saturation will change if no moisture is lost during the process.

A.8 Density.

A number of different "densities" are referred to in the text. As the use of the various terms can be extremely confusing the definitions for the terms as used are given below:-

A.8.1 Dry bulk density (kg/m³) - the mass of solid particles per unit volume of soil.

A.8.2 Apparent dry density (kg/m³) - the mass a cubic meter of solid material the density of which is measured by excluding the permeable voids but including the impermeable voids normal to the material. (i.e. the apparent relative mass density x 1 000).

The dry bulk density and the apparent dry density are properties of a soil mass. The relative mass densities listed below are however properties of the individual soil particles.

A.8.3 Bulk relative mass density - the ratio of the mass in air of a given volume of material (including permeable and impermeable voids normal to the material) at a stated temperature, to the mass in air of an equal volume of distilled water, at the same temperature. (TMH1, 1979).

A.8.4 Apparent relative mass density - the ratio of the mass in air of a given volume of material (excluding the permeable voids, but including the impermeable voids normal to the material) at a stated temperature to the mass in air of an equal volume of distilled water at the same temperature (TMH1, 1979).

The use of the word "relative" in the TMH definitions, bulk relative density and apparent relative density is unfortunate as it leads to confusion with the normal use of the concept "relative density" as defined in Appendix A.9.

The terms more commonly used for "bulk relative density" and "apparent relative density" are bulk specific weight and specific gravity respectively. These though they are the correct terms are not used in the test as the material properties were determined according to TMH1 and therefore the TMH1 nomenclature is used. Only the word "mass" is used in order to distinguish these properties from the concept "relative density" as defined in A.9.

A.9 Relative density. (D_r) (Holtz, 1972)

The relative density of a soil, where referred to in the text, is defined as follows:

- the state of compactness of a soil with respect to the loosest and densest states at which it can be placed by standard laboratory procedures (e.g. ASTM Designations: D4253 and D4254). It is expressed as the ratio of (i) the difference between the void ratio of a cohesionless soil in the loosest state and any given void ratio, to (2) the difference between its void ratios in the loosest and densest states.

Algebraically:

$$D_r = (\gamma_d - D_{min}) / (D_{max} - D_{min}) \times 100$$

or

$$D_r = (e_{max} - e) / (e_{max} - e_{min}) \times 100$$

where
D_{max} = dry density of soil in its densest state
D_{min} = dry density of soil in its loosest state
γ_d = dry density of compacted soil in the field
e_{max} = void ratio of soil in its loosest state
e_{min} = void ratio of soil in its densest state
e = void ratio of compacted soil in the field.

APPENDIX B - Amplitude characteristics of vibratory table.

The amplitude could not be varied by physical adjustment of the vibratory table used in the experimental work described in section 3. Measurements of amplitude of vibration were made by clamping a ballpoint pen to the table and obtaining a trace by drawing a sheet of paper attached to a clipboard past the vibrating pen. The trace was enlarged on a photocopier to increase the accuracy of measurement, and measured with a scale rule. Allowance was made for the factor by which the photocopies enlarged the trace, when calculating the amplitude. (cf Appendix A.3)

Three sets of amplitude determinations were made:-

i) For the situation with no load on the table, amplitude was measured on the pivot and the throw axes. A typical trace for each is shown in Fig B.1. The amplitude was 0.7mm on the pivot and 1.2mm on the throw axis.

The throw axis is defined as the central axis of the table parallel to the camshaft, while the pivot axis is the axis through the centre of the table at right angles to the camshaft.

ii) A series of measurements was taken with the mould and collar clamped to the table with different total loads on the table. Masses for which amplitude was measured included 15, 30, 50 and 110 kg. Typical traces are given in Fig B.2. For each mass a measurement was taken without steadying the mass by hand i.e. allowing it to "bounce", and a second reading was taken with the mass steadied. In all cases there was no soil in the mould. The measured amplitudes are given below in Table B.1

Total mass on table (kg)	Amplitude (mm)	
	mass steadied	mass unsteadied
15	0,59	1,23
30	0,64	1,55
50	0,71	1,53
110	0,67	1,72

Table B.1 Amplitude of vibration for different total masses

From the traces it can be seen that when the mass is not steadied the basic amplitude is modified by a secondary effect resulting from the "bouncing" out of phase of the mass. This results in a basic vibration, in addition to a superimposed effect which has a larger amplitude. If the mass is steadied, however, the superimposed effect is eliminated.

The amplitudes under the steadied surcharge masses remained approximately constant and the amplitude appeared not to be affected by an increase in total mass on the table of up to 110 kg.

iii) The amplitude was measured on the pivot axis with soil in the mould and a 50 kg surcharge mass. Measurements were taken as the soil was compacted under the vibration at 15 and 30 seconds and at 1, 2 and 4 minutes. The traces are given in Fig B.3. The amplitudes are tabulated below.

Time after start of vibration	Amplitude (mm)
15 sec	0,70
30 sec	0,53
1 min	0,35
2 min	0,35
4 min	0,35

Table B.2 Reduction in amplitude with time of vibration

The amplitude appears to be reduced as the soil becomes more dense. There exists therefore an interplay between the soil, the solid surcharge mass and the amplitude.

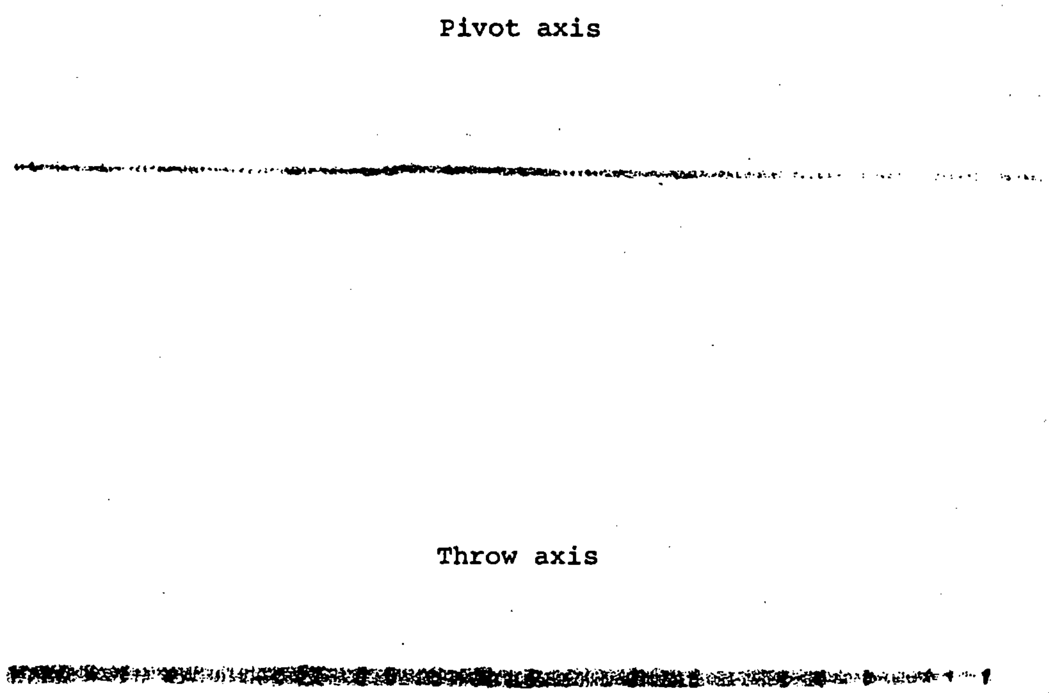
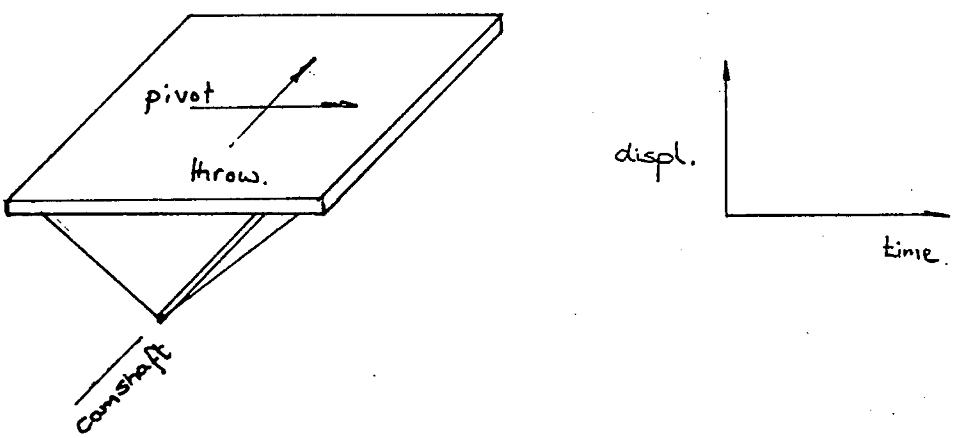
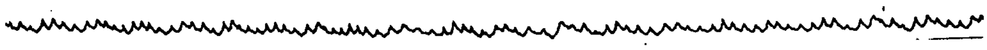


Fig B.1 Amplitude with zero load (actual size)



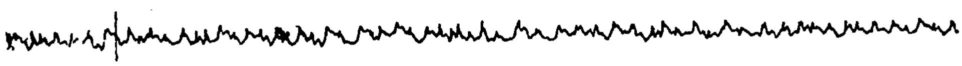
15 kg mass steadied by hand

15 kg mass unsteadied



30 kg mass steadied by hand

30 kg mass unsteadied

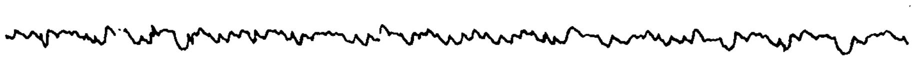


50 kg mass steadied by hand

50 kg mass unsteadied

110 kg mass steadied by hand

110 kg mass unsteadied



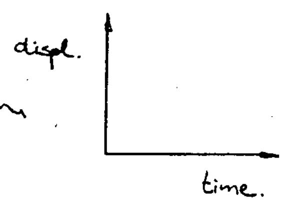


Fig B.2 Amplitude with no soil and different surcharge masses (actual size)

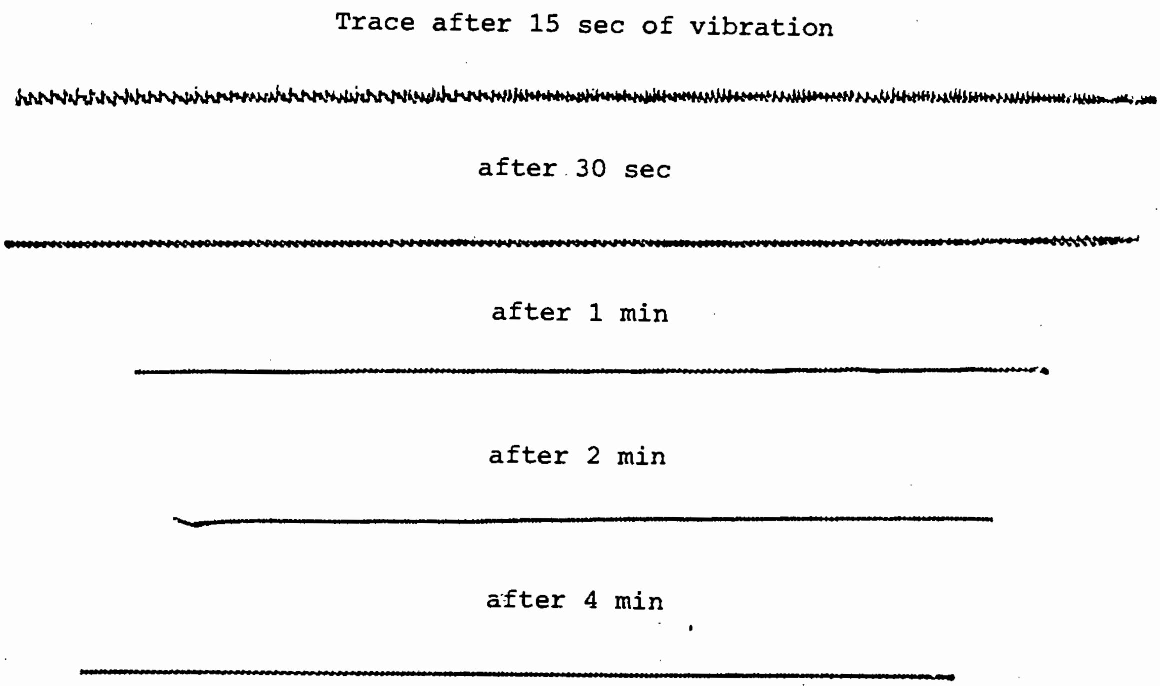
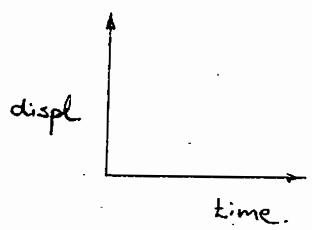


Fig B.3 Amplitude with 50 kg mass and with soil in mould



COURSES COMPLETED IN PARTIAL FULFILLMENT OF THE DEGREE

Courses/ Year	Course Title	Credit Value
CE 533 1980	Bridge Engineering	4
CE 516 1980	Prestressed Concrete	5
CE 532 1980	Advanced Foundation Design	5
CE 545 1983	Dam Design	5
CE 549 1983	Marine Pipelines	3
CE 5F2 1983	Contract Law	3
CE 5B6 1984	Frame Analysis	2
CE 5B8 1984	Plates and Shells	2
CE 5E7 1985	Deep Excavations	3
CE 5E4 1986	Rock Mechanics	3
CE 5H2 1986	Road Pavements	3
Thesis	Factors Influencing Laboratory Vibratory Compaction	20
	Total	58